Review of Structural Stability

Knightville Dam
Connecticut River Basin
Huntington, Massachusetts

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REVIEW OF STRUCTURAL STABILITY

KNIGHTVILLE DAM

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DEPARTMENT OF THE ARMY
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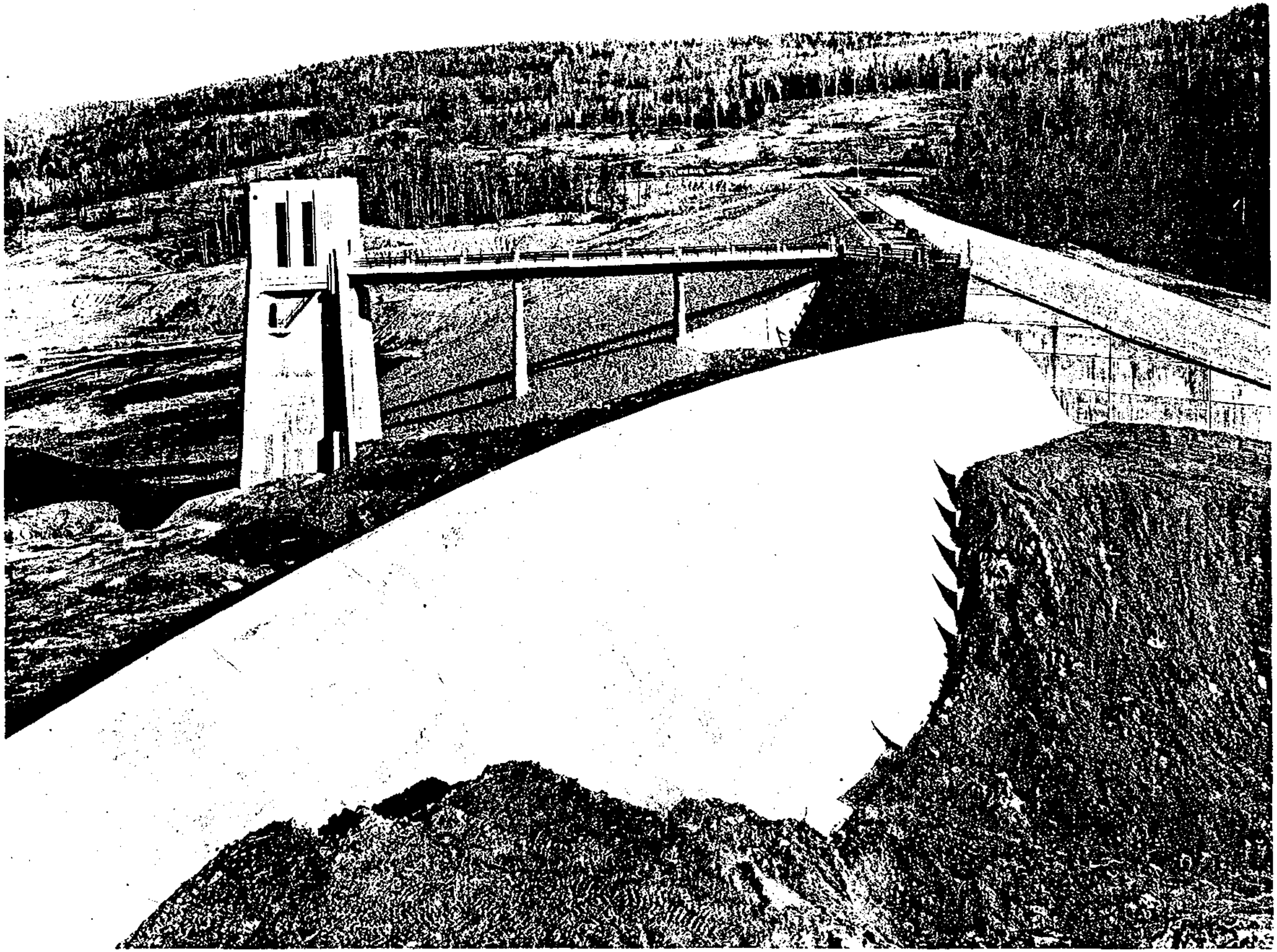

KNIGHTVILLE DAM MAY 1941

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NOTE: Appendix B is a separate volume

SUMMARY OF REPORT

A stability analysis of the principal concrete structures at Knightville Dam was performed to determine whether these structures satisfy current design criteria. The structural elements considered and the qualitative results of the analysis are as listed:

Structure	All Criteria Satisfied
Intake Tower	Yes
Bridge Piers	Yes
Spillway	No
Spillway Retaining Walls	Yes
Concrete Toe Wall	Yes

All of the concrete structures satisfy the prescribed requirements, except the spillway monoliths at maximum flood discharge condition. In this loading case the overturning stability criteria is not met for section B/24, the forth monolith from the east end, and section E/26 (respective percentages of base in bearing are 84, 62, 47). No remedial work is recommended for the following reasons:

1. The resultant does occur substantially within the base and is stable against overturning, although the criteria per se is not met.

2. The probability of full discharge is small.

3. As the spillway is cut from existing rock, there is little possibility of undermining due to erosion.

REVIEW OF STRUCTURAL STABILITY

KNIGHTVILLE DAM

PART I

GENERAL DESCRIPTION

1.1 Purpose

The objective of this study is to review the stability of the principal concrete structures, based upon current criteria in cases where the original design criteria were less conservative. This review is performed to comply with Corps of Engineers regulation ER-1110-2-100, Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures (28 February 1977).

1.2 Stability Criteria

The current stablity criteria by which this project is evaluated are contained in the following Corps of Engineers publications:

Engineering Manuals:

EM 1110-1-2101	Working Stress for Structural Design, 1 Nov 1963 (with Change 2, 17 Jan 1972)
EM 1110-2-2200	Gravity Dam Design, 25 Sept 1958 (with Change 2, 23 Nov 1960)
EM 1110-2-2400	Structural Design of Spillways and Outlets Works, 2 Nov 1964)
EM 1110-2-2501	Wall Design: Flood Walls, Jan 1948 (with Change 3, 18 June 1962)
EM 1110-2-2502	Retaining Walls, 29 May 1961 (with Change 3, 25 Jan 1965)

Engineer Technical Letter:

ETL 1110-2-256	Sliding Stability for Concrete Structures, 24 June 1981

Engineering Regulations:

ER 1110-2-1806	Earthquake Design and Analysis for Corps of Engineers Dams, 16 May 1983.

1.3 Pertinent References

Pertinent data, computations and drawings are contained in the following:

Analysis of Design Appendix A - Knightville Dam 1939

Anaylsis of Design - Knighville 1938

Periodic Inspection Report No. 1 - Knightville Dam December 1973

1.4 Project Description

Knightville Dam is located on the Westfield River about 4 miles north of the town of Huntington, Massachusetts. Construction of the dam and other structures was initiated in 1939 and completed in 1941. Recreational facilities were provided. The dam is of the hydraulic earth-fill type with a dumped rock shell. It has a top length of 1,200 feet and a maximum height above the stream bed of 160 feet. A curved concrete spillway, about 405 feet long, is located on rock in a natural saddle at the west end of the dam. The crest of the spillway is at Elevation 610; this is 20 feet below the top of dam to insure the dam against overtopping during the design discharge flood. Gated outlet works, founded on bedrock, are located under and at the west end of the dam embankment. The three gates are normally kept open and the reservoir empty. During time of flood, the gates are closed to temporarily store floodwaters in the reservoir.

The Knightville spillway was originally designed for a discharge of 91,000 cfs with a surcharge of 15 feet (EL. 625 NGVD) and a design freeboard of 5.0 feet. A review of the spillway design flood in the nineteen sixties using probable maximum precipitation from the hydro meteorlogical report #33 indicated a peak spillway discharge assuming outlet gates in operative, of 145,000 cfs with a maximum surcharge of 19.3 ft. (El. 629.3 NGVD) and a remaining freeboard of 0.7 feet. Further review in the nineteen seventies still using HMR #33 rainful but assuming the gates operable resulted in a revised spillway design discharge of 130,000 cfs with a maximum surcharge of 17.3 feet (EL. 627.3 NGVD) and a resulting freeboard of 2.7 feet.

1.5 Pertinent Hydraulic Data

The hydraulic data used for this review of structural stability are as follows:

Full Pool Condition - Reservoir at spillway crest elevation 610.0; downstream tailwater in outlet channel at elevation 463.0.

Design Discharge Condition - Reservoir at spillway design flood maximum surcharge elevation 629.3 with gates closed, elevation 627.3 with gates open, downstream tailwater in the outlet channel is at elevation 507.0.

1.6 Discussion of Analysis and Criteria

The principal structural elements analyzed for stability consists of the following:

(a) INTAKE TOWER

(b) BRIDGE PIERS

(c) SPILLWAY

(d) SPILLWAY RETAINING WALLS

(e) CONCRETE TOE WALL

Sliding stability of structures subjected to lateral loadings is assessed by the criteria presented in ETL 1110-2-256. The adequacy of sliding resistance is evaluated by determining a safety factor that is applied to the resisting shearing forces in a manner which places the forces acting on the structure in sliding equilibrium. For all of the structures analyzed, except for the spillway training walls, a minimum factor of safety of 2.0 is required for all conditions of loading when earthquake is not considered. For loading conditions when earthquake is considered, this factor of safety should exceed 1.3. The spillway training walls should have a factor of safety greater than 1.5 for all loading conditions.

The resistance to overturning is determined according to current criteria by the location of the resultant of vertical forces at the base. The resultant should be located within the middle third of the base for all conditions of loading when earthquake is not considered. For loading conditions where earthquake is considered, it is acceptable if the resultant stays within the base, provided that allowable foundation pressures are not exceeded. For retaining walls founded on rock, the resultant may be outside the middle third, but within the base, if foundation pressures are within allowable values and the factor of safety against sliding is adequate. There have been no significant changes in overturning criteria since the original computations were made.

Knightville Dam is loacted in Seismic Zone 2 (moderate damage) as shown on the Seismic Zone Map of Contiguous States, included with ER 1110-2-1806. Therefore, this analysis takes into account earthquake forces induced by accelerations equal to 0.10g. Earthquake forces were not considered in the original design computations.

In accordance with EM 1110-2-2200, the seismic forces applied to this stability analysis are as follows:

(a) Inertia force Pe_1 due to acceleration of the structure, acting through the center of gravity in any direction. $Pe_1 = 0.10W$, where W is the weight of the structure.

(b) Inertia force Pe_2 induced by the impoundment of water. This force is computed using Westergaard's formula and the following parameters are used throughout: acceleration equal to 0.10g, period of vibration equal to 1 second, $C = 51$ lbs/ft^3. (When a structure was completely surrounded by water, the virtual mass method was applied per EM-1110-2400 P. 27.)

(c) Dynamic earth pressure, as outlined in EM 1110-2-2502, is accounted for by adding the weight of backfill between a sloping wall and a vertical plane through the heel to the wall weight for computation of inertia force Pe_1.

No vertical acceleration is considered in this analysis. Uplift is assumed to be unaffected by earthquake accelerations.

The uplift pressure at any point under a structure is the tailwater pressure plus the pressure measured as an ordinate from tailwater to the hydraulic gradient between the upstream and downstream pool. Uplift pressure is considered to act over 100 percent of the base area. The uplift considered in the original 1939 design was only 50 percent of these values.

Ice pressure of 10,000 pounds per linear foot of structure is applied in this analysis in accordance with EM 1110-2-2200. Ice pressure was considered to be 1,000 pounds per linear foot in the original design.

Wind pressure of 30 pounds per square foot is used in this stability investigation and in the original computations.

1.7 <u>Discussion of Foundation and Foundation Parameters</u>

All of the structural elements considered in this stability analysis, except for the service bridge abutment, are founded on rock. As described in the Definite Project Report - Knightville Dam, the bedrock formation underlying the dam site consists of quartzitic and mica schist.

The bedding is steeply inclined with angles of inclination varying between 60^0 west and 80^0 west. The strike of the bedding is approximately north-south.

Mechanical weathering, chiefly frost action, has affected the upper portions of the formation near the surface by opening small cracks along the bedding planes. In quartzitic schist varieties, these cracks become less prominent or entirely disappear within varying depths of from 5 to 15 feet. In all other respects, the rock is structurally sound.

All concrete structures analyzed are shown on the contract drawings to be founded upon solid rock. Excavation to sound rock was estimated to be approximately 4 feet deep in the spillway area. Sealing cracks and small fissures in the rock beneath retaining walls and the concrete was were required during construction.

Allowable bearing pressures on the foundation materials described above are not given in the original design computations. An allowable bearing pressure of 35 tons per square foot on bedrock is assumed in this analysis.

The shear strength of the foundation materials is computed using the Mohr-Coulomb failure criteria as described in ETL 1110-2-256. Throughout this analysis, the critical potential failure surface for sliding stability is assumed to be a single plane at the interface of concrete structure and foundation material.

All of the structural elements analyzed, except for the spillway weir and the service bridge piers are subjected to lateral forces induced by earth backfill. Earth pressures acting on the concrete toewall and spillway retaining walls, which are founded on rock, are in accordance with EM 1110-2-2502.

Foundation parameters used for this analysis are as follows:

(a) Allowable bearing pressure on bedrock = 35 tons per square foot (assumed value).

(b) Shear at interface between rock and concrete = 75 pounds per square inch (based on ACI 318-71, composite concrete, allowable bond shear stress for clean and intentionally roughened contact surfaces without mechanical anchorages).

(c) Coefficient of fricitonal resistance = 0.7 (concrete on rock).

(d) Coefficient of active earth pressure = 0.27 (based on internal angle of friction = 35^0 and corrected, where necessary, to account for sloping backfills).

(e) Coefficient of at-rest earth pressure = 0.5 and corrected, where necessary, to account for sloping backfills.

1.8 Method of Computation

Stability of all structures was investigated by manual calculations.

PART II

RESULTS OF THE ANALYSIS

2.1 INTAKE TOWER

The intake tower is located at the upstream end of the tunnel directly above the transition section and is founded on solid rock. In plan, the tower measures approximately 35 feet by 46 feet at the top and has variable dimensions within its height, including diagonal counterforts extending up the tower at the four corners. The total height of the tower from the roof of the transition section to the floor of the operating house is 138 feet. The downstream face of the tower is cast against a rock cut for a height of approximately 67 feet, leaving a free height of the tower of approximately 71 feet.

The tower was analyzed for stability at three levels; Elevations 545, 526.5, and 477 (on rock). From the loading cases listed in EM 1110-2-2400, Section 3-07.c, entitled "Stability of Gate Structure at Upstream End", the applicable cases are as follows:

Case I. Reservoir empty. Wind load to produce most severe foundation pressures.

Case II. Gate structure with all gates open. Reservoir at spillway crest. Ice pressure. Uplift. Water surface inside structure drawn down to hydraulic gradient with all gates open.

Case III. Similar to Case II, except that gate structure operating with one outside gate closed, others open.

Case IV. Gate structure with gates closed. No flow in conduits. Reservoir at spillway crest. Ice pressure. Uplift. Structure full of water upstream from closed gates.

Case V. Reservoir raised to spillway design flood level for whichever of preceding Cases II, III, or IV is most critical. No ice pressure.

Case IA, IIA, IIIA, and IVA. Same as Case I, II, III, and IV, respectively, with earthquake load added.

At the upper levels, Elevations 545 and 526.5, the bending and shearing stresses in concrete are well within allowable limits.

Without seismic loading, the stability requirements for overturning are satisfied except for case II and IV. For these cases, with maximum ice pressure on one side, the resultant falls outside of the kern with 78 and 88 percent of the base remaining in bearing. Considering the tower base embedment into rock below El.545.0 and the bearing of the diagonal counterforts against rock providing additional overturning resistance, the resultant will be within the kern.

With seismic loading, two cases find the resultant just within the base (with high theoretical base pressures and two cases find the resultant just outside of the base. However, the probability of an earthquake occuring when the reservoir is at spillway crest (Knightville dam is a dry bed reservoir) is very small. Considering the tower base embedment into rock and the bearing of the diagonal counterforts, the actual resultant will be well within the base with associated bearing pressures reduced.

The results of the stability analysis for the intake tower are contained in Table I. Under the specified loading cases, the intake tower is stable and no modification or strengthening is required.

TABLE I
STABILITY ANALYSIS OF INTAKE TOWER

Section (1)	Loading Case	LOCATION OF RESULTANT In Middle Third	LOCATION OF RESULTANT In Base	Sliding Factor of Safety	Percent of Base in Bearing	Bearing Pressure On Rock KIPS/S.F. MAXIMUM	Bearing Pressure On Rock KIPS/S.F. MINIMUM
EL. 545	I	YES	YES	135	100	14.5	9.4
EL. 545	I A	YES	YES	24	100	20.8	3.2
EL. 526.6	I	YES	YES	135	100	13.0	11.3
EL. 526.6	I A	NO	YES	20	73	33.0	0
EL. 477.0	II	NO	YES	43	78	21.1	0
EL. 477.0	II A	NO	YES	12	27	60.3	0
EL. 477.0 H	II A	NO	YES	10	4	464.0	0
EL. 477.0 P	II	YES	YES	55	100	15.0	1.4
EL. 477.0 P	II A	NO	YES	12	42	39.2	0
EL. 477.0 P H	II A	NO	YES	10	16	103.8	0
EL. 477.0	III	YES	YES	18	100	12.7	2.8
EL. 477.0	III A	NO	YES	8	56	27.8	0
EL. 477.0 H	III A	NO	YES	7	29	53.4	0
EL. 477.0 P	III	YES	YES	54	100	15.5	0
EL. 477.0 P	III A	NO	YES	12	20	77.4	0
EL. 477.0 P H	III A	NO	NO	10	0 (1.1ft outside)	--	--
EL. 477.0	IV	NO	YES	8	88	17.1	0
EL. 477.0 H	IV A	NO	NO	5	0 (1.2ft outside)	--	--
EL. 477.0	IV A	NO	YES	5	21	65.3	0
EL. 477.0	V/II	YES	YES	--	100	13.4	4.0
EL. 477.0	V/III	YES	YES	29	100	13.3	2.3
EL. 477.0	V/IV	YES	YES	10	100	8.2	7.4

(1) Unless noted "P", stability is analyzed in upstream-downstream direction. "P" indicates stability analysis perpendicular to direction of flow. "H" indicates hydrodynamic forces included in analysis.

2.2 SERVICE BRIDGE PIERS

Two intermediate piers of reinforced concrete, founded on rock, support the service bridge connecting the intake tower with the dam. The three-span service bridge, whose longitudinal superstructure members consist of two steel plate girders, has a total length of 210 feet. The design loading is AASHTO H-15.

Loading cases considered are those specified below.

Case I. Dead Load reaction of bridge. No water. Wind.

Case IA. Dead Load reaction of bridge. No water. Earthquake.

Case II. Dead Load reaction of bridge. Water level at Spillway EL. 610.00 with uplift.

Case IIA Dead Load reaction of bridge. Water level at Spillway EL. 610.00 with uplift. Earthquake

Case III. Dead load reaction of bridge. Water at maximum discharge EL. 629.3

The free standing Pier No. 1 was analyzed. Pier No. 2, built integrally with a retaining wall, is more stable and therefore, did not require a separate analysis.

Stability was checked at Elevation 558, which is the average depth of concrete foundation embedded in a sloping rock surface, as shown on contract drawings. The top of pier is approximately 65 feet above this reference line. In calculating dead load reaction for the maximum discharge condition (EL. 629.3) the bridge deck was assumed to be fully submerged as the roadway elevation is only a little more than one foot higher than the probable maximum flood. Factor of safety against uplift during flood is 2.2.

Wind loading of 30 psf was applied at 30^0 to the longitudinal axis of the bridge to give the maximum lateral load to be resisted by the minimum pier cross section. Ice forces, acting all around the pier, would not affect the stability of the pier.

The minimum factor of safety against sliding based only on frictional resistance is 24, greater than the required factor of safety of 1.5.

For Pier 1, the resultant is within the kern of the base for Loading Cases I, II, and III. For Loading Case IIA with uplift on the pier and earthquake forces, the resultant falls outside of the base. To prevent overturning of the pier a horizontal reaction at the bridge deck through bearings on the pier is necessary. The reaction computed is relatively small, only 840 pounds. This force would have to be shared by two expansion and two fixed bearings with eight 1-1/4" Ø anchor bolts, and transmitted to the entire bridge structure through the deck. It is unlikely that any horizontal movement of the top of the pier would occur and it would be limited to a 2-inch gap in the expansion dam. Therefore, no remedial measures are needed to improve the stability of the service bridge piers. Table II contains the results of the analysis.

TABLE II

STABILITY ANALYSIS OF SERVICE BRIDGE PIERS

Loading Case	LOCATION OF RESULTANT: In Middle Third	LOCATION OF RESULTANT: In Base	Sliding Factor of Safety	Percent of Base in Bearing	Bearing Pressure on ROCK KIPS/S.F.: MAXIMUM	Bearing Pressure on ROCK KIPS/S.F.: MINIMUM
I	YES	YES	111	100	9.0	3.1
IA	NO	YES	24	41	15.9	0
II	YES	YES	-- (1)	100	4.7	4.7
IIA	NO	NO	24	0	-	-
III	YES	YES	-- (1)	100	3.5	3.5

(1) Summation of horizontal forces is equal to zero, and therefore, factor of safety against sliding is undefined.

2.3 Spillway.

The ogee-shaped concrete spillway is approximately 400 feet long at the crest. The structure is divided into fourteen concrete monoliths, typically 30 feet long and separated by expansion joints with copper waterstops. The central part consists of eight monoliths, varying in height from approximately 40 to 70 feet. The spillway crest is at Elevation 610. The toes of these monoliths are embedded in rock to a depth of at least 6 feet along the downstream side.

The three monoliths at the east end of the spillway were built to the initial crest elevation of 600 and later raised to the final elevation of 610. The total height is about 35 feet, the embedment of toe in rock is a minimum of 4 feet. The horizontal construction joint at Elevation 600 is reinforced with vertical steel dowels along the upstream face and with inclined dowels on the downstream side. The last monolith at the east end of the spillway is anchored into the retaining wall by means of horizontal steel dowels.

The four small monoliths at the west end of the spillway were initially built to Elevation 600 and then raised to Elevation 610. These monoliths are only 16 feet high, with embedment of toe in rock to a minimum of 3 feet. There are five rows of steel anchors drilled into the rock abutment and dowels at both faces in the horizontal construction joint at Elevation 600 (see sheet No. 26 Appendix A). Contract drawings do not indicate horizontal dowels into rock at the first monolith.

The width of the spillway approximately equals it height. As the monoliths are not interconnected by shear keys, each of them has to be stable by itself under any loading condition. Four typical monoliths were analyzed.

Loading cases applied are in accordance with EM 1110-2-2200, Section 3-01. Applicable were Case II - normal operating, IV - flood discharge, and VI - normal operating with earthquake.

The hydrologic data used for this spillway are the following:

Loading Cases II and VI - Full Pool Condition (pool at spillway crest, minimum tail water):

Energy gradient at spillway (ft. msl)	610.00
Tail-water energy gradient	463.0

Loading Case IV - Design Discharge Condition (reservoir at peak level of probable maximum flood):

Energy gradient at spillway (ft. msl)	627.3 (gates open)[1]
Tail-water energy gradient (ft. msl)	510.0
Tail-water water surface (ft. msl)	507.0

The values for the factors of safety against sliding, bearing pressures and location of resultant for each monolith analyzed are shown in Table III. For Sections A/24, B/24, and E/26, and the fourth monolith from the east end were investigated (Sections refer to contract drawings). Deviations from required criteria, when only a few percent, were considered insignificant.

Under Loading Case II, with ice forces, the resultant was found to be within the middle third of the base. Under Load Case VI, the resultant is always within the base. Spillway Section E/26 above the construction joint at Elevation 600 was analyzed for Loading Cases IV and VI and was found to be stable.

The overturning stability criteria for Loading Case IV is not satisfied for Section B/24, the fourth monolith from the east end, and Section E/26 (respective percentages of base is bearing are 84, 62, 47). No remedial work is suggested for the following reasons:

1. The resulting does occur substantially within the base and is stable against overturning, although the criteria per se is not met.

2. The probability of full discharge is small.

3. As the spillway is cut from existing rock, there is little possibility of undermining due to erosion.

The minimum factor of safety against sliding was found to 3.8; the maximum bearing was found to 9.70 KIPS/S.F. Both meet current criteria.

[1]See pg. 2

TABLE III

STABILITY ANALYSIS OF SPILLWAY

Section	Loading Case	LOCATION OF RESULTANT In Middle Third	LOCATION OF RESULTANT In Base	Percent Base In Bearing	Resistance to Sliding Factor of Safety	Bearing Pressure on Rock Maximum KIPS/S.F.	Bearing Pressure on Rock Minimum KIPS/S.F.
Central B/24	II	Yes	Yes	100	4.9	7.15	0.97
	IV	No	Yes	84 (3.8'outside)	3.8	8.81	0
	VI	No	Yes	97	3.9	8.41	0
West End A/24	II	Yes	Yes	100	19.9	1.19	.55
	IV	Yes	Yes	100	14.8	1.36	0.18
	VI	Yes	Yes	100	30.7	.98	.76
East End E/26	II	No	Yes	94	7.8	5.6	0
	IV	No	Yes	47 (7.2'outside)	5.5	9.7	0
	VI	No	Yes	98	7.2	5.3	0
SECTION ABOVE C.J. AT EL 600.0							
	IV	No	Yes	99	16.5	1.20	0
	VI	Yes	Yes	100	43.8	.85	.63
East End Fourth Monolith at Concrete Base	II	Yes	Yes	100	6.0	5.97	2.83
EL. 559	IV	No	Yes	62 (7.3' outside)	3.9	8.52	0
	VI	Yes	Yes	100	4.9	6.28	0
East End Fourth Monolith at 10 Feet	II	Yes	Yes	100	10.0	5.92	1.58
Below Concrete	IV	Yes	Yes	100	4.5	3.77	3.47
Base in Rock Elevation 549	VI	Yes	Yes	100	7.3	5.21	2.29

2.4 Spillway Retaining Walls.

There are two retaining walls near the dam: one separates the earth-fill embankment from the spillway weir, and the other protects the downstream toe of the dam at the river channel from erosion at the outlet. Both walls are concrete gravity sections. The latter will be discussed in the next section, Concrete Toe Wall.

The retaining wall starts at one pier of the service bridge, includes the bridge abutment, connects with the east end of the spillway and extends downstream about 150 feet from spillway. The maximum height of this wall is about 55 feet, with a corresponding width of 40 feet, and the minimum height is 10 feet at the south end. The full length of the wall is founded on rock with embedment 2 to 3 feet deep.

In accordance with EM 1110-2-2502, the retaining wall was analyzed for at rest and active earth pressures, with no fill or water in front of the wall, with the following exception:

(a) Upstream wall, during flood, with water on all sides of the wall.

Uplift pressures assumed are 100 percent of hydrostatic head at the heel and zero at the toe.

Loading cases considered were as follows:

Case I	-	Normal water level (maximum Elevation 610).
Case IA	-	Normal water level plus earthquake.
Case II	-	Floodwater level, Elevation 629.3.
Case III	-	Water level on both sides up to Elevation 610.
Case IIIA	-	Water level on both sides plus earthquake.

The latter two cases, III and IIIa, are applicable to walls on the upstream side of the spillway.

The tabulated values of factors of safety and bearing pressures for each wall section analyzed are shown in Table IV. With earthquake forces, the vertical resultant may be located outside of the middle third of the base. For such cases, the percentages of the width of base which will be in bearing are calculated and none of the bearing pressures is excessive. All wall sections have adequate stability under all loading cases considered.

TABLE IV

STABILITY ANALYSIS OF SPILLWAY RETAINING WALLS

Section	Loading Case	LOCATION OF RESULTANT In Middle Third	In Base	Sliding Factor of Safety	Percent Base in Bearing	Bearing Pressure on Rock KIPS/S.F. Maximum	Minimum
D-25	I	Yes	Yes	6.1	100	10.3	0.7
(60 Feet High)	II	Yes	Yes	12.4	100	6.0	1.6
	I-A	No	Yes	4.0	53	20.7	0
C-25	I	Yes	Yes	6.2	100	9.1	0.5
(60 Feet High)	II	Yes	Yes	10.5	100	5.3	1.4
	III	Yes	Yes	8.4	100	6.3	0.6
	I-A	No	Yes	4.4	61	15.7	0
	III-A	No	Yes	5.3	60	11.4	0
F-25 High	I	Yes	Yes	6.8	100	7.2	0.8
(45 Feet High)	I-A	No	Yes	4.8	63	12.8	0
F-25 Low	I	Yes	Yes	9.8	100	5.2	0.4
(31 Feet High)	I-A	No	Yes	6.7	58	9.6	0

2.5 Concrete Toe Wall

This retaining wall of concrete gravity section protects the downstream toe of the dam at the river crossing from erosion by the outlet flow. It was designed for hydrostatic head and lateral rock pressure. Having a total length of 232 feet, this wall varies in height from a maximum of 76 feet to a minimum of 5 feet. The wall consists of five different monoliths separated by expansion joints. The top elevation starts at Elevation 547.5 feet at the west end and slopes down to Elevation 500.6 feet at the other end. The contract drawings show that the base of the toe wall is built on sound rock excavated several feet below the original rock line. In plan, this wall follows a circle with a radius of 156 feet.

The analysis of stability was done for three different monoliths. The sections were analyzed as gravity walls for the following loading cases:

- Case I-1 - Full pool, water at the rear of toe wall at Elevation 503 feet.
- Case I-2 - Maximum flood, water at both sides of toe wall at Elevation 507 feet.
- Case II-1A - Loading consists of Case I-1, as outlined above, plus earthquake forces.

According to EM 1110-2-2502, Sec. 4.e., a vertical resultant location outside the middle third is acceptable when at-rest lateral earth pressures are used. Accordingly, the use of middle third criteria for gravity walls on rock analyzed for active earth pressure produces an adequate factor of safety for at rest pressure. Therefore, this stability analysis was done using "active" pressure produced by the rock backfill ($\phi = 45^0$, $K_a = 0.19$). To allow for the effect of the backfill sloping upward, the horizontal force was applied at 0.45 times the height. The acceptable location of the resultant is within the middle third, except for earthquake loadings where the resultant must fall within the base.

The tabulated values of factors of safety and bearing pressures for each monolith analyzed are shown in Table V. None of these pressures is excessive and factors of safety calculated are greater than the minimum required. Therefore, all wall sections can be considered to be stable under all loading condition.

TABLE V

STABILITY ANALYSIS OF CONCRETE TOE WALL

Section	Loading Case	LOCATION OF RESULTANT		Sliding Factor of Safety	Length of Base in Bearing (ft)	Bearing Pressure on Rock KIPS/S.F.	
		In Middle Third	In Base			Maximum	Minimum
Top Elevation 539	I-1	Yes	Yes	4.9	100	14.4	0.9
(76 Feet High)	I-2	Yes	Yes	4.3	100	9.8	3.7
	II-1A	No	Yes	4.0	67	23.0	0
Top Elevation 528	I-1	Yes	Yes	6.1	100	10.7	2.5
(65 Feet High)	I-2	Yes	Yes	10.4	100	7.1	3.8
	II-1A	No	Yes	4.2	78	17.1	0
Top Elevation	I-1	Yes	Yes	12.0	100	5.8	0.4
(28 Feet High)	I-2	Yes	Yes	22.9	100	3.4	1.4
	II-1A	No	Yes	8.1	70	8.8	0

2.6 CONCLUSIONS

All of the Knightville Dam concrete structures analyzed meet the prescribed requirements, except the spillway monoliths at maximum flood discharge condition. In this loading case the overturning stability criteria is not satisfied for section B/24, the fourth monolith from the east end, and section E/26 (Respective Base percentages in bearing are 84, 62, 47). No remedial work is suggested for the following reasons:

1. The resulting does occur substantially within the base and is stable against overturning, although the criteria per se. is not met.

2. The probability of full discharge is small.

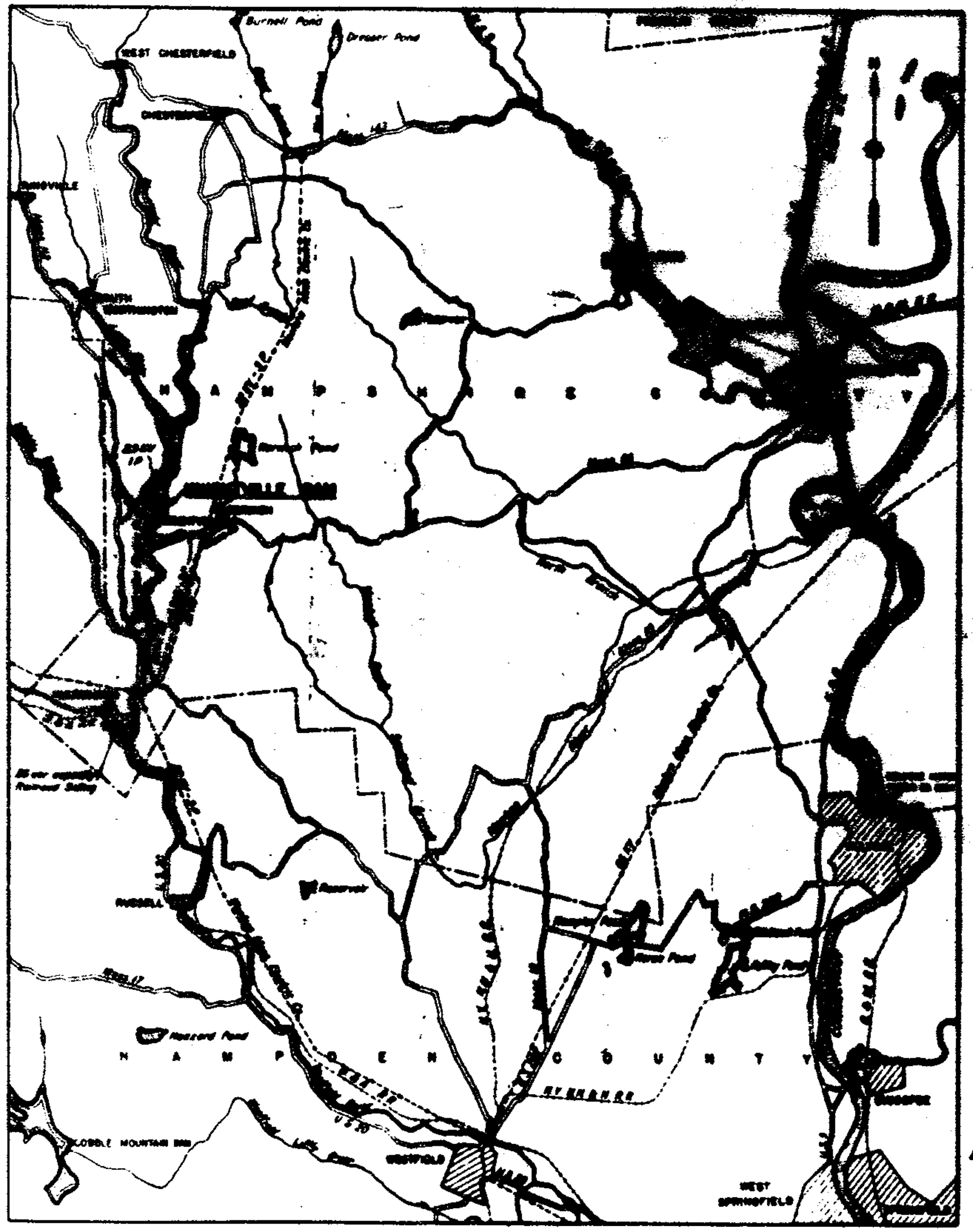
3. As the spillway is cut from existing rock, there is little possibility of undermining due to erosion.

In order to strictly satisfy current overturning criteria for the spillway weir, the location of the resultant for the maximum flood discharge condition would have to be improved. This could be accomplished by installling a system of post-tensioned rock anchors. The construction cost for this modification is estimated to be $800,000.

APPENDIX A

SELECTED RECORD DRAWINGS

Drawing No.		Title
CT-1-	Sh. No. 1	Project Location & Index
CT-1-	Sh. No. 6	General Plan
CT-1-1234	Sh. No. 11	Outlet Works
CT-1-1247	Sh. No. 12	Assembly
CT-1-1265	Sh. No. 13	Intake Transition No. 1
CT-1-	Sh. No. 17	Intake Tower Sections No. 1
CT-1-1280	Sh. No. 20	Intake Tower Sections No. 4
CT-1-1232	Sh. No. 28	Bridge Piers
CT-1-1261	Sh. No. 24	Spillway - Detail Plan & Sectionsq
CT-1-1284	Sh. No. 25	Spillway - Retainging Wall No. 1
CT-1-1315	Sh. No. 26	Spillway - Retaining Wall No. 2
CT-1-1278	Sh. No. 27	Spillway - Retaining Wall No. 3
CT-1-1235	Sh. No. 29	Concrete Toe Wall



VICINITY MAP

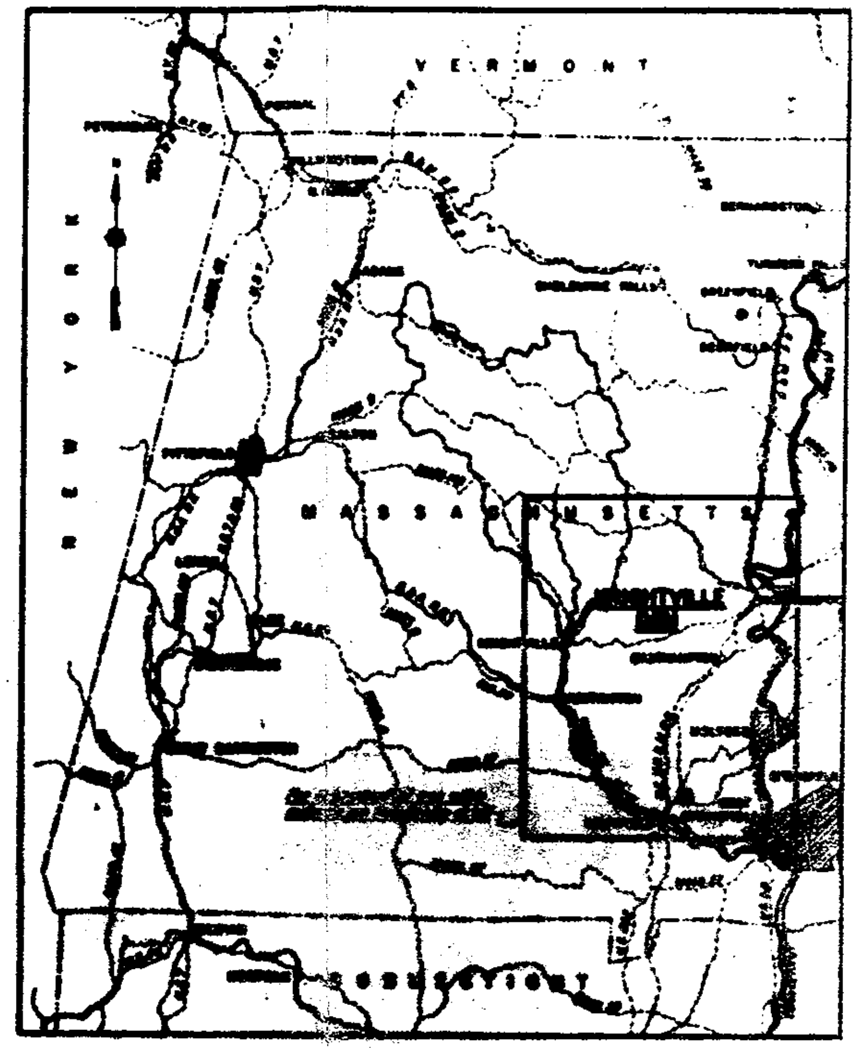
SCALE

LEGEND

HIGHWAYS
RAILROADS

INDEX TO DRAWINGS

SHEET NO.	TITLE
1	PROJECT LOCATION AND INDEX
2	[illegible]
3	PLAN OF SUBSURFACE EXPLORATIONS
4	RECORD OF SUBSURFACE EXPLORATION NO. 1
5	RECORD OF SUBSURFACE EXPLORATION NO. 2
6	GENERAL PLAN
7	EMBANKMENT DETAILS NO. 1
8	EMBANKMENT DETAILS NO. 2
9	EMBANKMENT DETAILS NO. 3
10	FOUNDATION TREATMENT
11	OUTLET WORKS
12	[illegible]
13	INTAKE TRANSITION NO. 1
14	INTAKE TRANSITION NO. 2
15	INTAKE TRANSITION NO. 3
16	INTAKE TRANSITION NO. 4
17	INTAKE TOWER SECTIONS NO. 1
18	INTAKE TOWER SECTIONS NO. 2
19	INTAKE TOWER SECTIONS NO. 3
20	INTAKE TOWER SECTIONS NO. 4
21	INTAKE TOWER SECTIONS NO. 5
22	INTAKE TOWER SECTIONS NO. 6
23	[illegible] PORTAL
24	SPILLWAY
25	RETAINING WALL NO. 1
26	RETAINING WALL NO. 2
27	RETAINING WALL NO. 3
28	[illegible]
29	CONCRETE TOE WALL
[illegible]	[illegible]
68	[illegible]
69	OPERATING HOUSE - PLANS
70	OPERATING HOUSE - ELEVATIONS AND SECTIONS NO. 1
71	OPERATING HOUSE - ELEVATIONS AND SECTIONS NO. 2
72	OPERATING HOUSE - DETAILS
73	OPERATING HOUSE - STRUCTURAL STEEL
74	TILE GAGES
75	ACCESS ROAD
▲ 76	PENSTOCK TRANSITION DETAILS
77	INTAKE TRANSITION - STEEL REINFORCEMENT NO. 7
78	INTAKE TOWER - STEEL REINFORCEMENT NO. 10
79	INTAKE TOWER - STEEL REINFORCEMENT NO. 11
80	PENSTOCK ENTRANCE - STEEL REINFORCEMENT
81	BAR SCHEDULE NO. 10
82	BAR SCHEDULE NO. 11
△ 34A	BORROW AREAS



LOCATION MAP

SCALE

LEGEND

7

CONNECTICUT RIVER

KNIGHTVILLE DAM

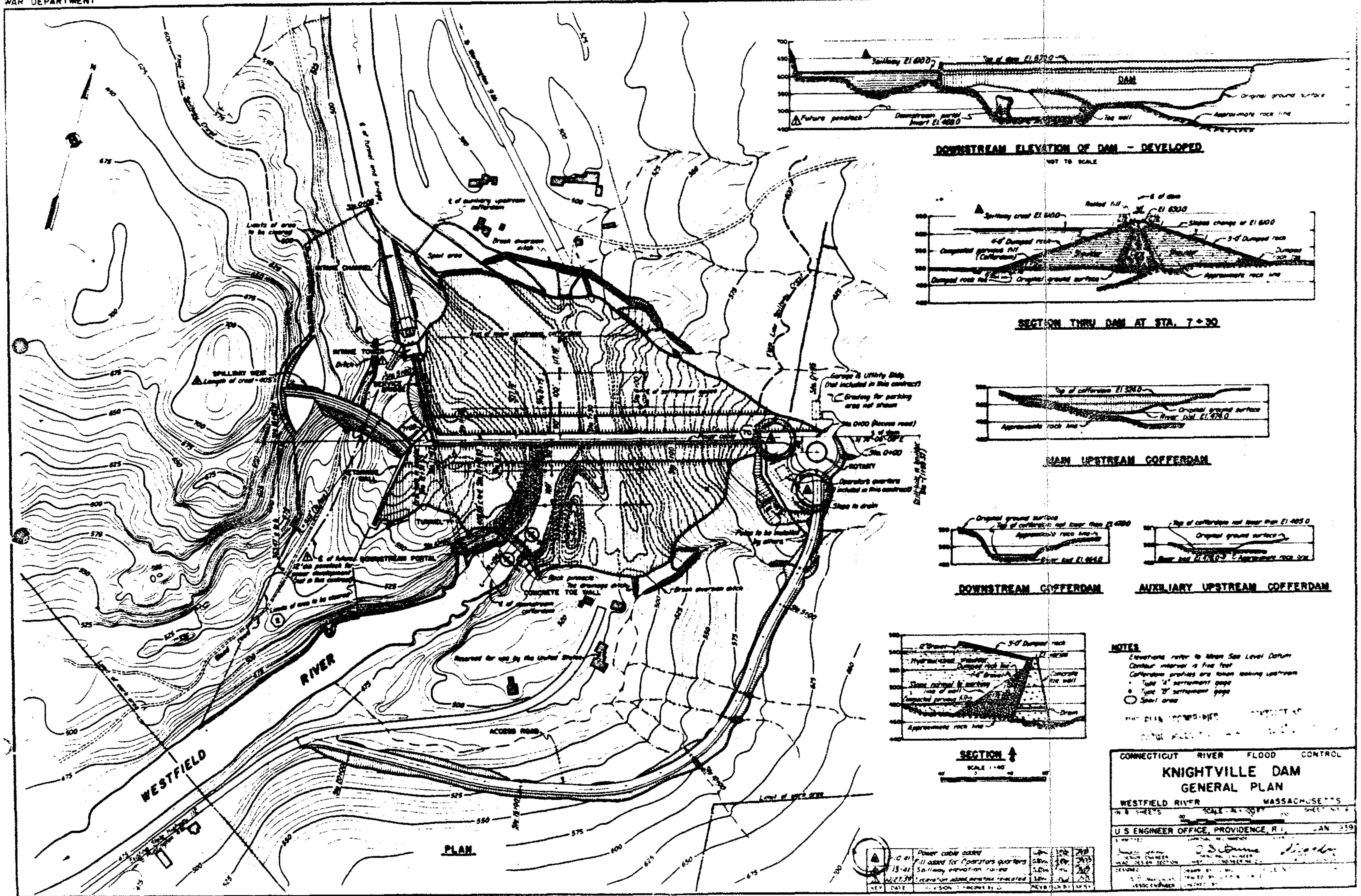
PROJECT LOCATION AN[illegible]

WESTFIELD RIVER

U.S. ENGINEER OFFICE

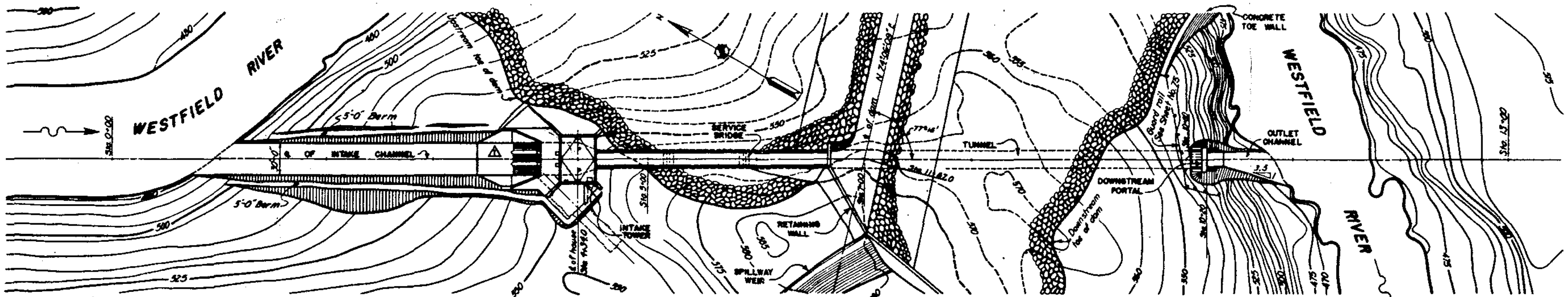
WAR DEPARTMENT

CORPS OF ENGINEERS,

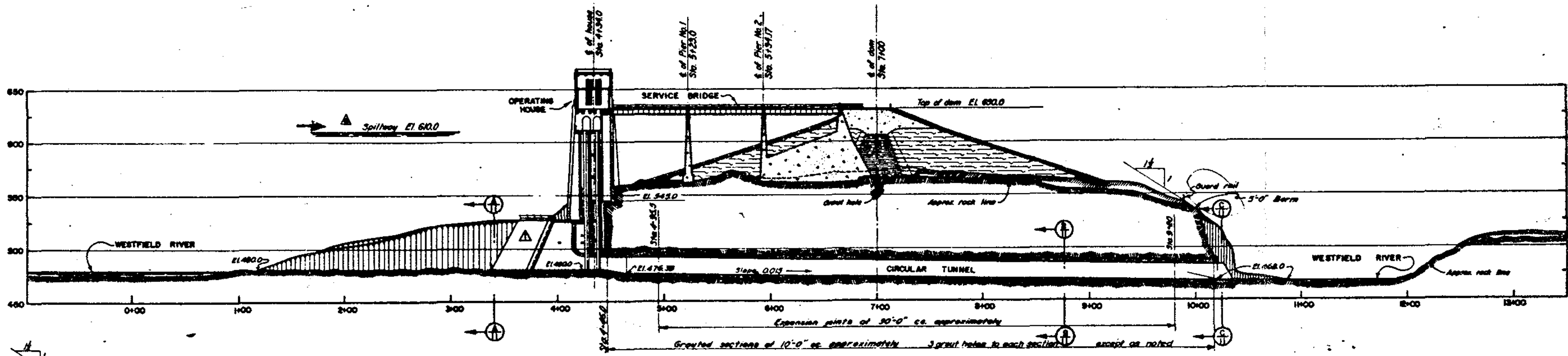


PRINTED

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY



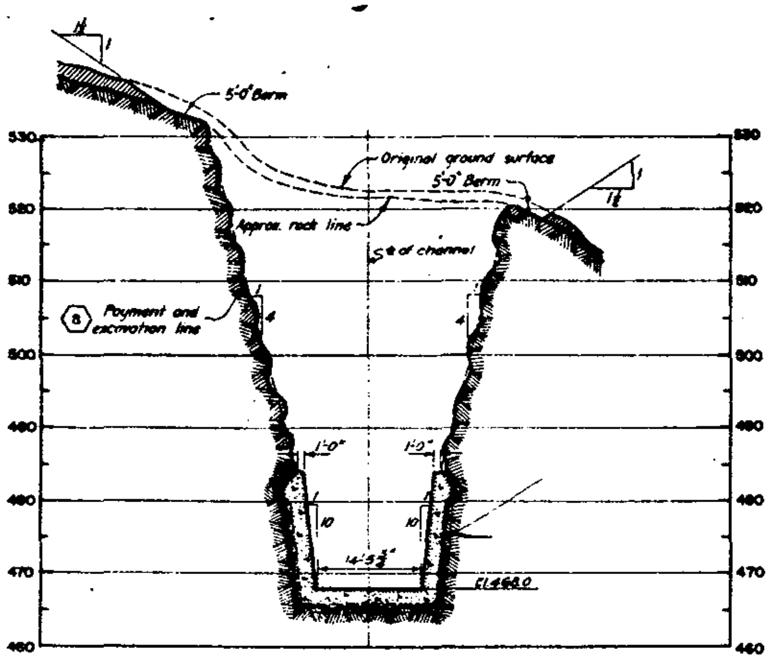
PLAN
SCALE 1 IN. = 40 FT.



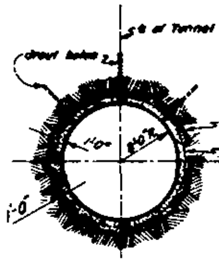
SECTION ON CENTER LINE OF TUNNEL
SCALE 1 IN. = 40 FT.

NOTES

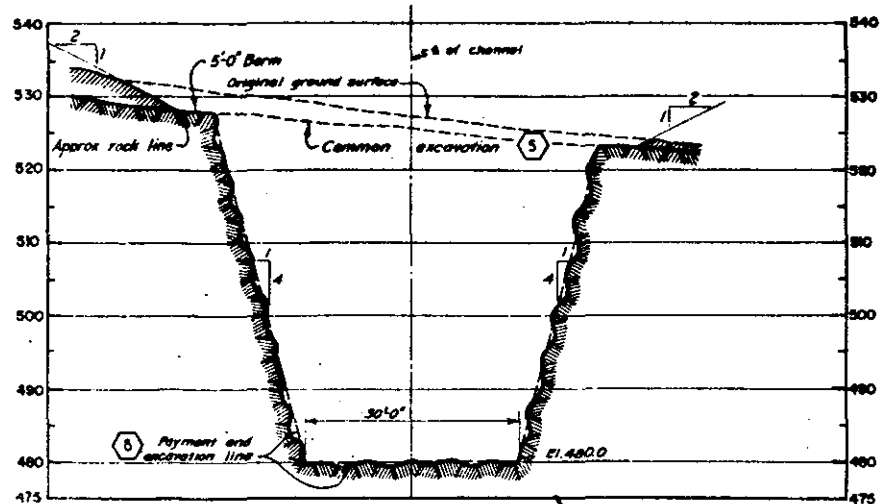
Elevations refer to Mean Sea Level Datum.
The location of the downstream portal is subject to change.
All expansion and field joints in the tunnel shall have copper water stops.
Contour interval — 5 feet.
Figures in hexagons indicate item numbers under which payment will be made.



SECTION C/11
SCALE 1 IN. = 10 FT.



SECTION B/11
SCALE 1 IN. = 10 FT.



SECTION A/11
SCALE 1 IN. = 10 FT.

KEY	DATE	REVISION (Indicated by △)	REV. BY	CK. BY	AP. BY
△	1-13-41	Spillway elevation raised			
△	10/27/39	Piers and channel walls extended			

THIS PLAN ACCOMPANIES CHANGE ORDER NO. 8 CONTRACT NO. W 639 Eng- 848

CONNECTICUT RIVER FLOOD CONTROL
KNIGHTVILLE DAM
OUTLET WORKS
WESTFIELD RIVER, MASSACHUSETTS
IN 82 SHEETS SCALE: 1 IN. = 40 FT. SHEET NO. 11
U.S. ENGINEER OFFICE, PROVIDENCE, R.I. JAN. 1939

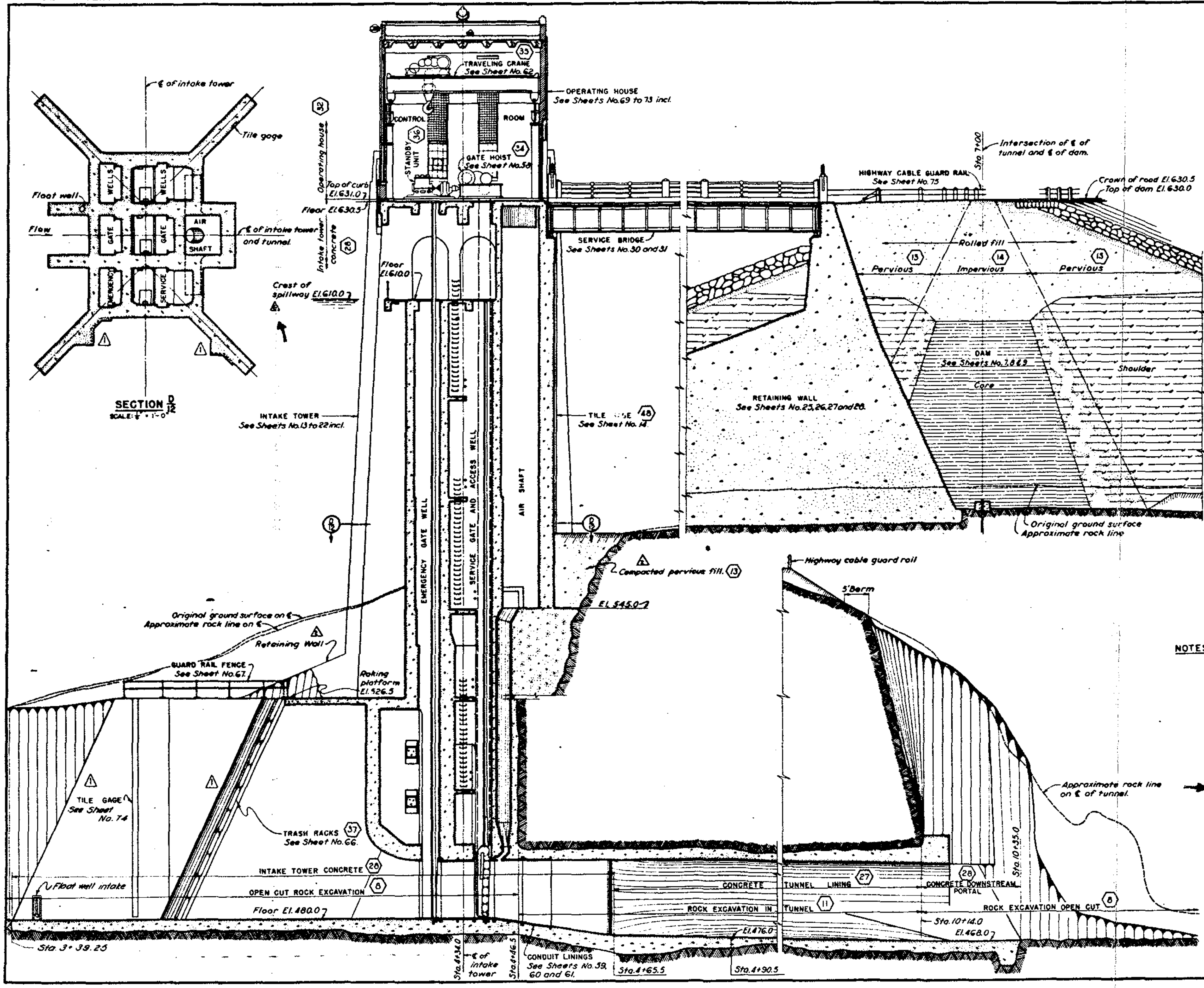
SUBMITTED: SENIOR ENGINEER, HEAD, DESIGN SECTION
APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV.
APPROVED: LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER
DESIGNED: ASSOC. ENGINEER
DRAWN BY: J.S.M.
TRACED BY: G.F.B. & I.C.W.
CHECKED BY: C.K.
FILE NO. CT-1-1234

11

PRINTED JAN 18 194[illegible]
F.C. Engineering Div.

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



NOTES

Elevations refer to Mean Sea Level Datum.
Figures in hexagons indicate item numbers under which payment will be made.

THIS PLAN ACCOMPANIES CHANGE ORDER NO. 8

CONTRACT NO W 69 Eng- 848

KEY	DATE	REVISION (indicated by △)	REV BY	CH BY	AP BY
△	1-13-41	Spillway elevation raised			
△	3/21/40	Retaining wall, rock cut added.			
△	10/27/39	Piers and walls extended			

CONNECTICUT RIVER FLOOD CONTROL

KNIGHTVILLE DAM
ASSEMBLY

WESTFIELD RIVER MASSACHUSETTS

IN 82 SHEETS SCALE: 1/8 IN. = 1 FT. SHEET NO. 12

U.S. ENGINEER OFFICE, PROVIDENCE, R.I. JAN. 1939

SUBMITTED: SENIOR ENGINEER, HEAD, DESIGN SECTION
APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV.
APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DESIGNED: ASSOC. ENGINEER
DRAWN BY: J.McL.
TRACED BY: J.T.
CHECKED BY:

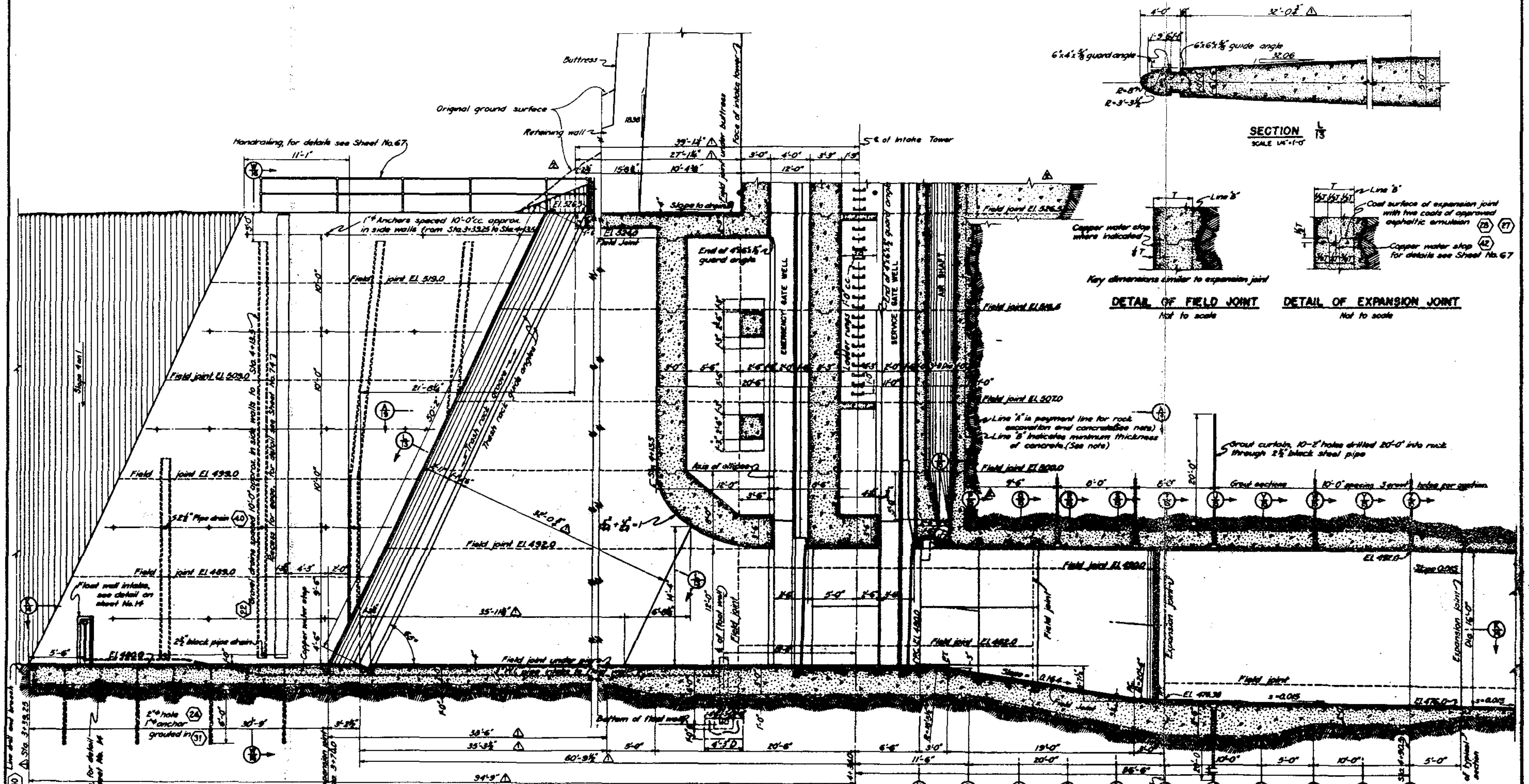
FILE NO. CT-1-1247

12

PRINTED ... JAN 13 1941

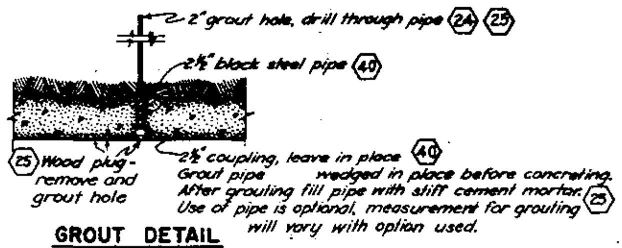
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



SECTION ON ℄ OF INTAKE TOWER AND TRANSITION

SCALE 1/4" = 1'-0"



GROUT DETAIL

NOTES

All concrete in intake tower and transition between Stations 3+39.25 and 4+65.5 shall be Class A and paid for under Item No. 28

Concrete in tunnel between Stations 4+65.5 and 10+14.0 shall be Class A and paid for under Item No. 27

Copper water stops shall be used in all joints downstream from Sta. 4+13.5 and below El. 526.5 as indicated

All outside corners of concrete shall be chamfered as directed by the contracting officer.

For details of miscellaneous metals see Sheets No. 65 to 67 inclusive.

For steel reinforcement see Sheets No. 33 to 39 inclusive and sheet No. 43.

Size of gate and grout recesses are tentative and definitely determined when the gates are detailed.

Figures in hexagons indicate item number under which payment made.

Rock overbreak paid for under Item No. 58.

Rock overbreak beyond the "A" line shall be backfilled with Class A concrete up to

NOTES (Continued)

indicated elevations. Payment for the excess Class A concrete made under Item No. 59.

Rock surfaces as of December 1, 1939, after initial excavation, are not shown throughout on all sections. They are indicated only in places where there are changed conditions in the concrete construction.

RECORD DRAWINGS

KEY	DATE	REVISION (Indicated by △)			
△	3/18/40	Retaining wall, rock cut added			
△	10/27/39	Piers and walls extended			

CONNECTICUT RIVER FLOOD CONTROL

KNIGHTVILLE DAM

INTAKE TRANSITION NO. 1

WESTFIELD RIVER, MASSACHUSETTS

IN 83 SHEETS — SCALE 1/4 IN. = 1 FT. — SHEET NO. 13

U. S. ENGINEER OFFICE, PROVIDENCE, R.I. JAN. 1939

SUBMITTED: SENIOR ENGINEER, HEAD, DESIGN SECTION

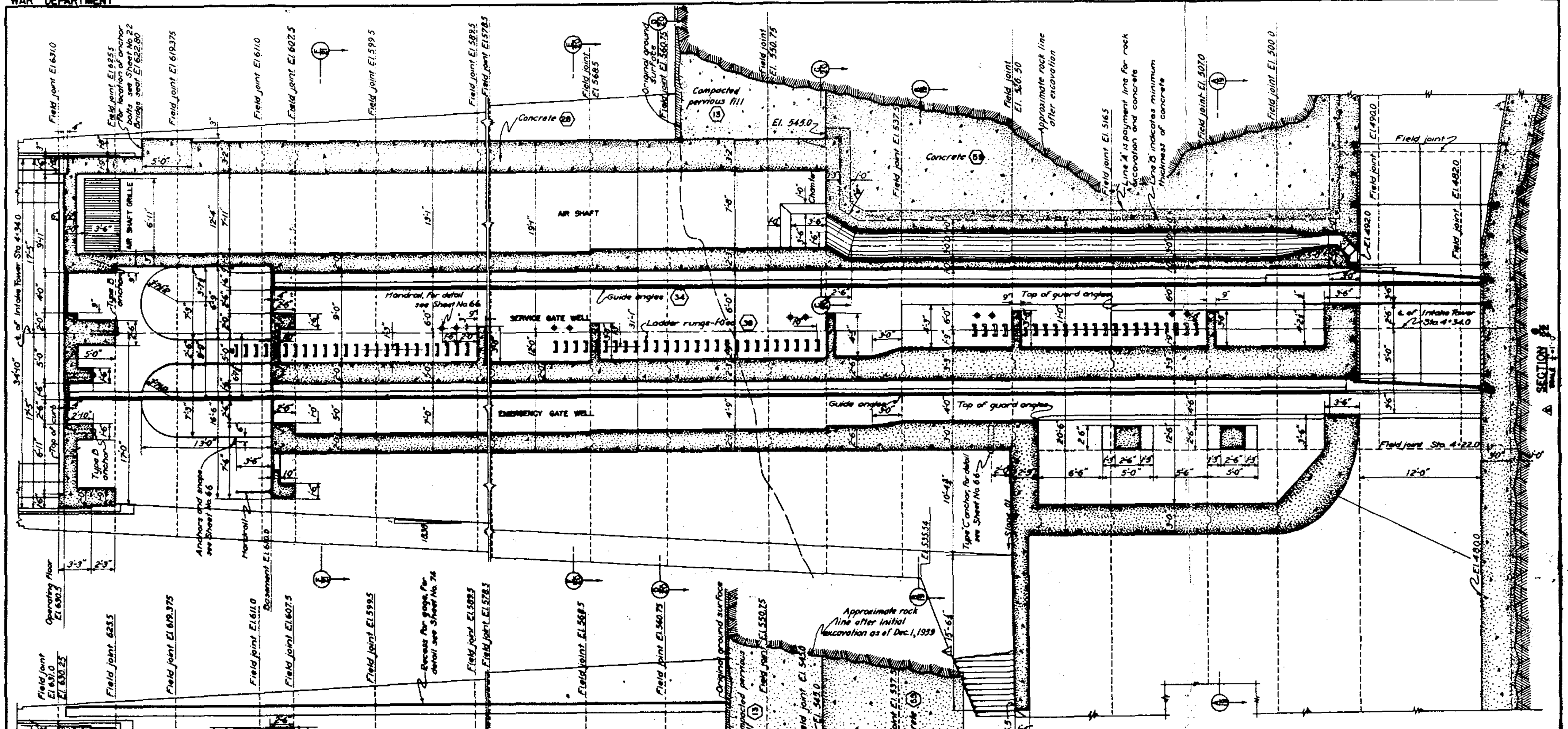
APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV.

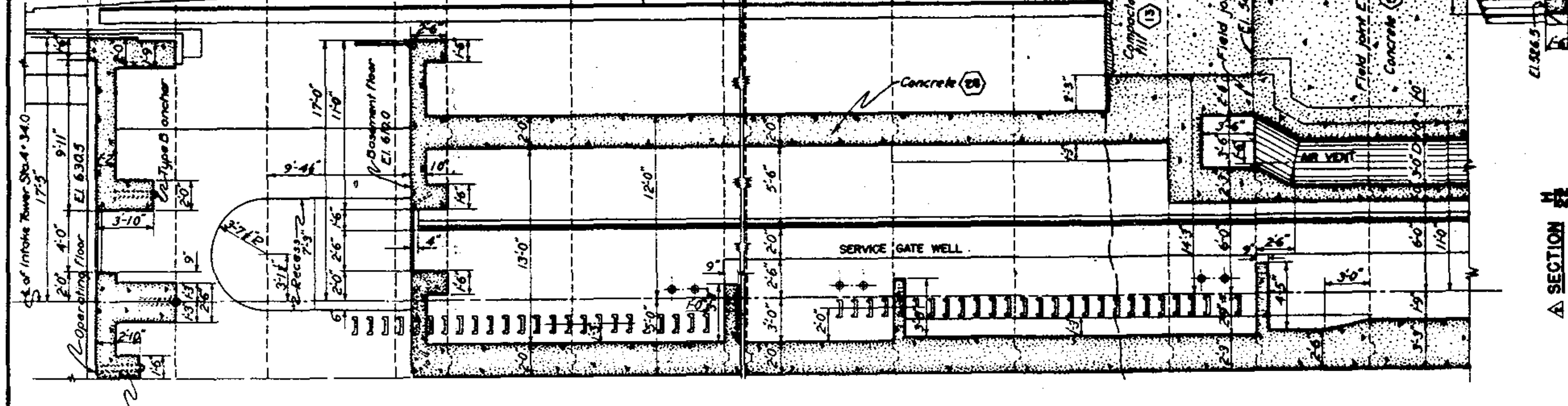
APPROVED: LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

DESIGNED:

DRAWN BY: R.S.H.
TRACED BY: K.S.H.
CHECKED BY:

FILE NO. CT-1-1265





SECTION 18/18A
SCALE 1/4"=1'-0"

NOTES

All concrete in the intake tower shall be Class A and paid for under Item No. 28.

Copper water stops shall be used in field joints below El. 526.5 as indicated.

All outside corners of concrete shall be chamfered as directed by the contracting officer.

Figures in hexagons indicate item numbers under which payment made.

For details of miscellaneous metals see Sheets Nos. 65, 66, & 67.

For steel reinforcement see Sheets No. 39 to 47 inclusive.

Rock overbreak beyond the "A" line paid for under Item No. 58.

Rock overbreak beyond the "A" line shall be backfilled with Class A concrete up to specified elevations. Payment for the excess Class A concrete made under Item No. 59.

RECORD DRAWINGS

CONNECTICUT RIVER FLOOD CONTROL

KNIGHTVILLE DAM
INTAKE TOWER
SECTIONS NO. 1

WESTFIELD RIVER — MASSACHUSETTS

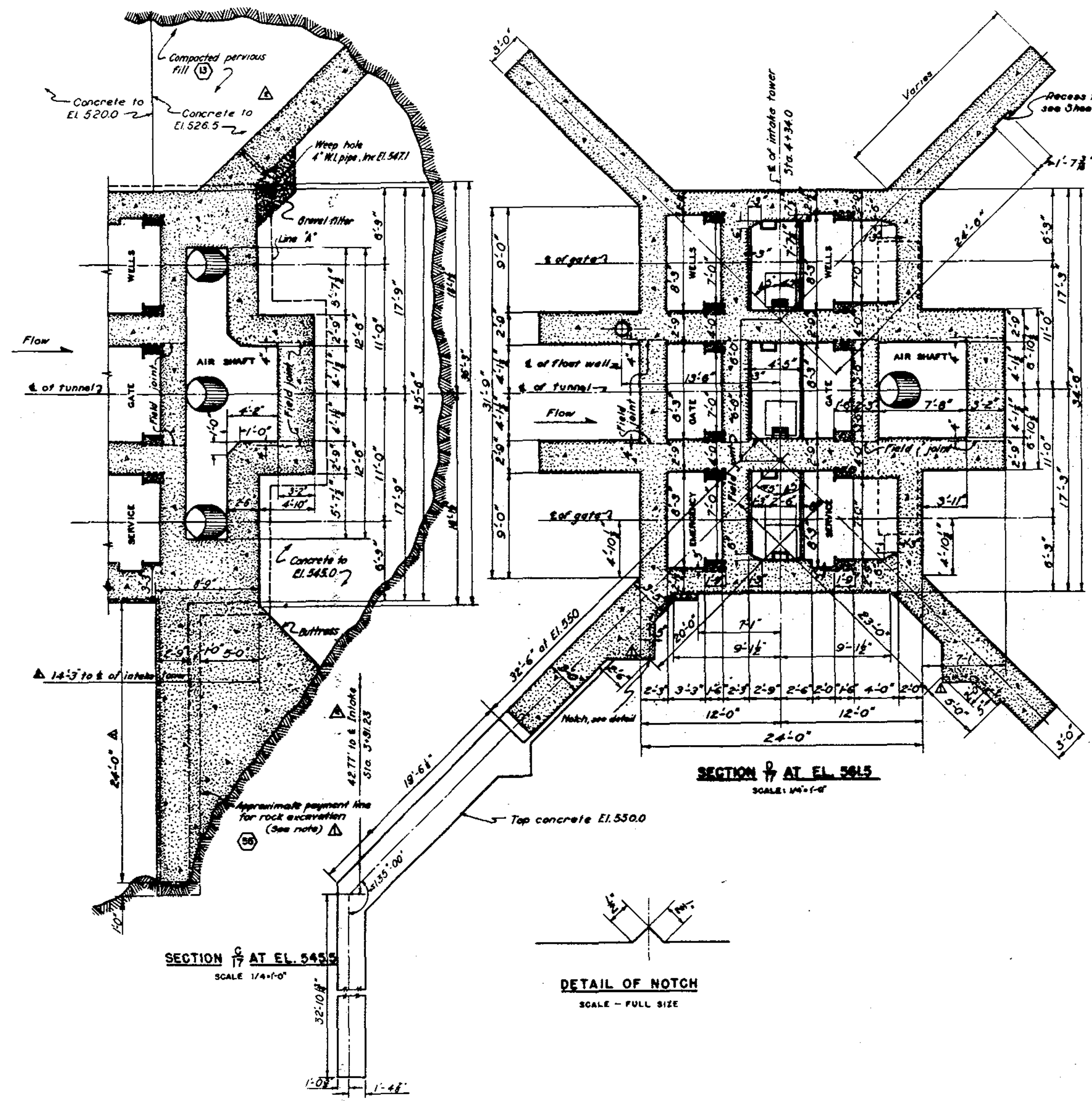
IN 83 SHEETS — SCALE: 1/4 IN.=1 FT. — SHEET NO. 17

U.S. ENGINEER OFFICE, PROVIDENCE, R.I. JAN. 1939

SUBMITTED: SENIOR ENGINEER, HEAD, DESIGN SECTION
APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV.
APPROVED: COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DESIGNED: ASSOC. ENGINEER
DRAWN BY: R.S.M.
TRACED BY: E.J.B.
CHECKED BY:
FILE NO. CT-1-1260

KEY	DATE	REVISION (indicated by △)	REV. BY	CK. BY	AP. BY
△	3/21/40	Rock cut and fill changes			
△	10/27/39	Raking platform extended			



SECTION $\frac{C}{17}$ AT EL. 545.5

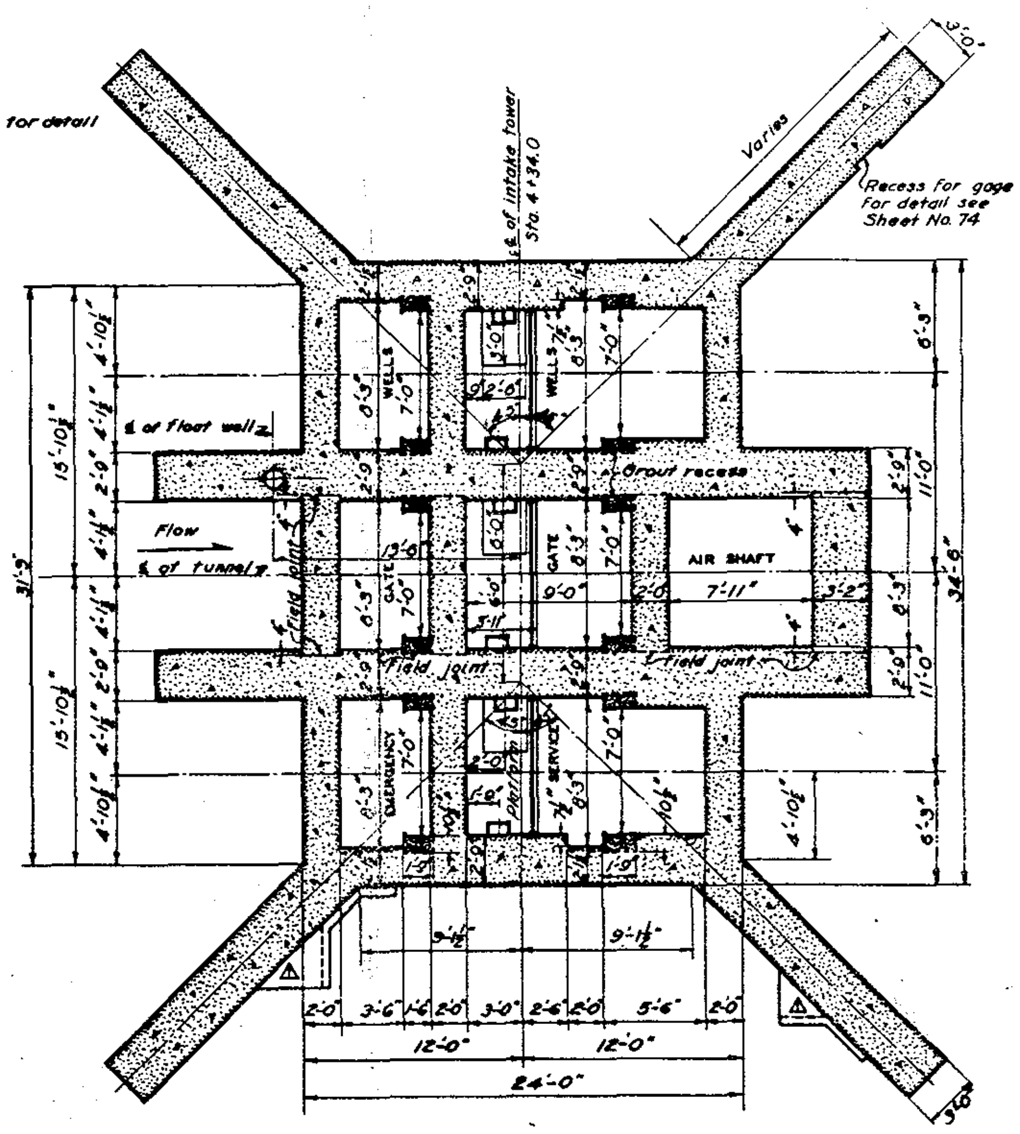
SCALE 1/4"=1'-0"

SECTION $\frac{D}{17}$ AT EL. 561.5

SCALE: 1/4"=1'-0"

DETAIL OF NOTCH

SCALE – FULL SIZE



SECTION $\frac{E}{17}$ AT EL. 570.0

SCALE: 1/4"=1'-0"

RECORD DRAWINGS

NOTES

For general notes applying to details on this sheet see Sheet No. 17

△ Payment line for rock excavation in the penstock intake and tunnel determined in the field and payment made under Items No. 55 and 56 respectively.

△ All concrete in the penstock intake shall be Class A and paid for under Item No. 57

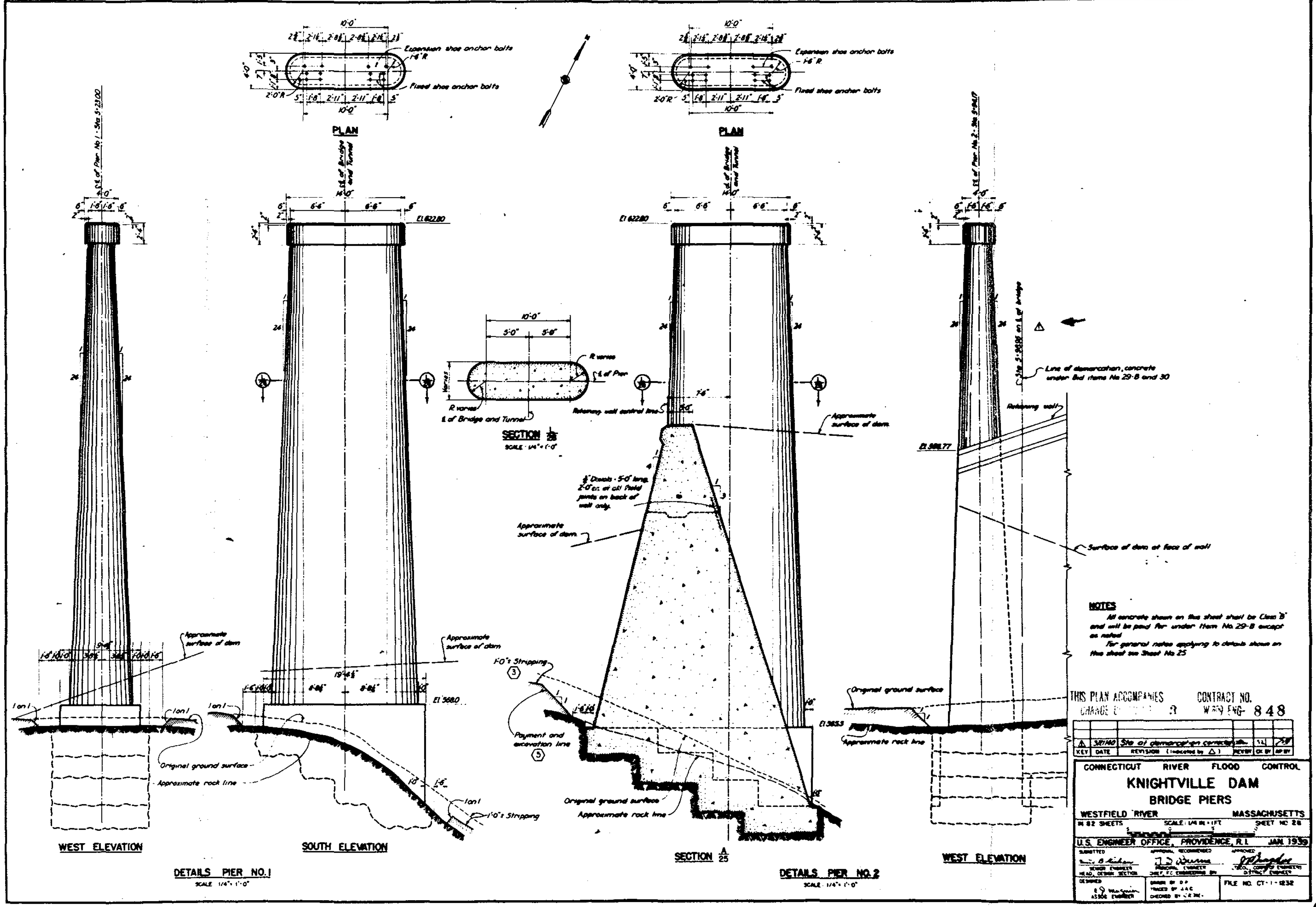
KEY	DATE	REVISION (Indicated by △)	REV. BY	CK. BY	AP. BY
	Sept 1949	Final field correction			
△	3/21/40	Rock cut added, wall thickened	[illegible]	[illegible]	[illegible]
△	10/27/39	Notes added	A.D.M.	[illegible]	[illegible]
△	10/27/39	Excavation, Wall and joint supports added	G.D.M.	[illegible]	[illegible]

CONNECTICUT RIVER FLOOD CONTROL
KNIGHTVILLE DAM
INTAKE TOWER
SECTIONS NO. 4
WESTFIELD RIVER MASSACHUSETTS

IN 83 SHEETS — SCALE: 1/4 IN. = 1 FT. — SHEET NO. 20

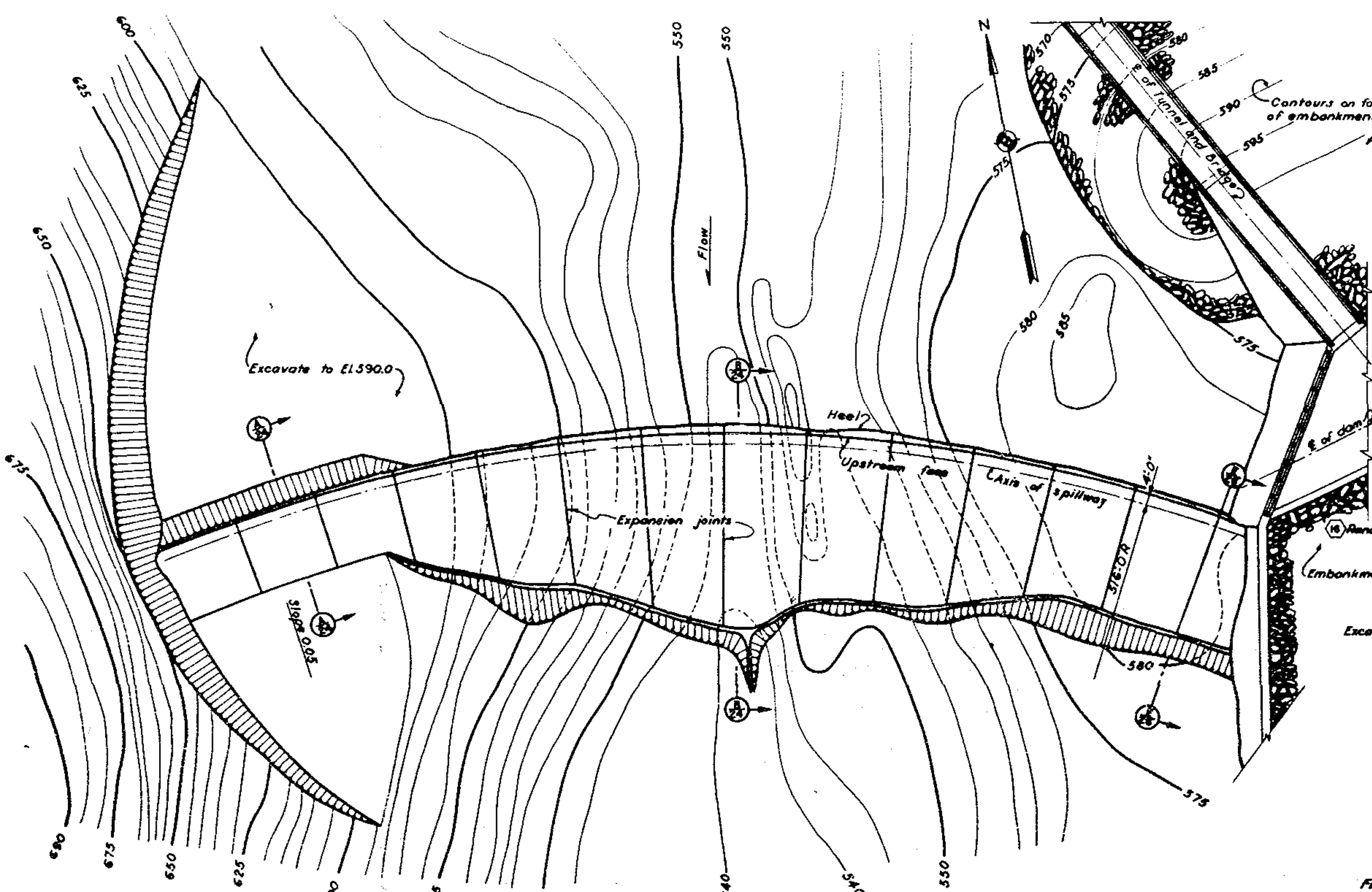
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1939

SUBMITTED:	APPROVAL RECOMMENDED:	APPROVED:
SENIOR ENGINEER HEAD, DESIGN SECTION	PRINCIPAL ENGINEER CHIEF, F.C. ENGINEERING DIV.	LT. COL. CORPS OF ENGINEERS DISTRICT ENGINEER
DESIGNED: G. D. [illegible] ASSOC. ENGINEER	DRAWN BY: R.S.M. TRACED BY: R.M.F. CHECKED BY: A.W.H.	FILE NO. CT-1-1280


PLAN
Expansion shoe anchor bolts
Fixed shoe anchor bolts
PLAN
SECTION
Line of demarcation, concrete under Bid items No. 29-B and 30
Retaining wall control line
Approximate surface of dam
Retaining wall
Surface of dam at face of wall
Approximate surface of dam
Original ground surface
Approximate rock line
Original ground surface
Approximate rock line
Payment and excavation line
1'-0" Stripping
WEST ELEVATION
SOUTH ELEVATION
SECTION A/25
WEST ELEVATION
DETAILS PIER NO. 1
SCALE 1/4" = 1'-0"
DETAILS PIER NO. 2
SCALE 1/4" = 1'-0"
NOTES
All concrete shown on this sheet shall be Class "B" and will be paid for under Item No. 29-B except as noted.
For general notes applying to details shown on this sheet see Sheet No. 25
THIS PLAN ACCOMPANIES CHANGE ORDER 3
CONTRACT NO. W-49 ENG- 848
KEY | DATE | REVISION (Indicated by △) | REV BY | CK BY | AP BY
CONNECTICUT RIVER FLOOD CONTROL
KNIGHTVILLE DAM
BRIDGE PIERS
WESTFIELD RIVER MASSACHUSETTS
IN 82 SHEETS
SCALE: 1/4 IN = 1 FT
SHEET NO 28
U.S. ENGINEER OFFICE, PROVIDENCE, R.I. JAN. 1939
SUBMITTED
SENIOR ENGINEER HEAD, DESIGN SECTION
APPROVAL RECOMMENDED
PRINCIPAL ENGINEER CHIEF, F.C. ENGINEERING DIV
APPROVED
LT. COL., CORPS OF ENGINEERS DISTRICT ENGINEER
DESIGNED
ASSOC. ENGINEER
FILE NO. CT-1-1232

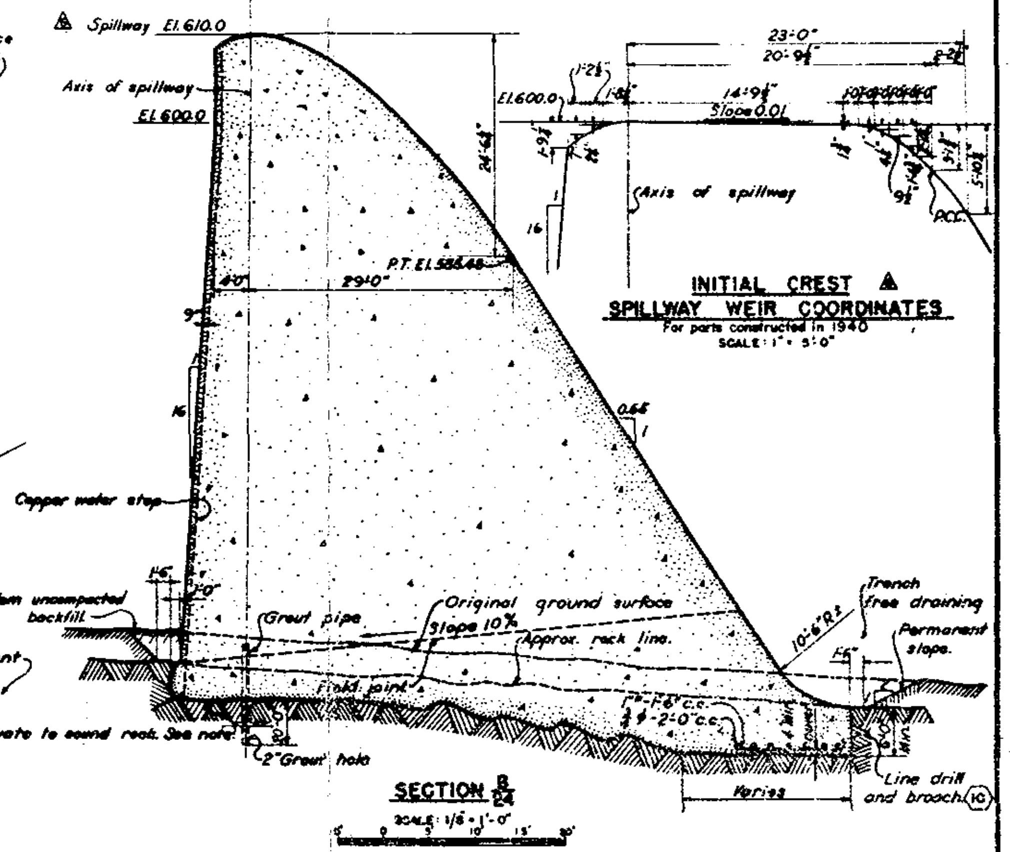
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



▲ PLAN

SCALE: 1" = 20'-0"



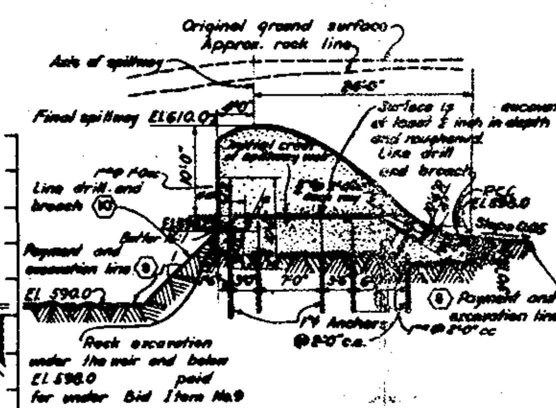
SECTION B/24

SCALE: 1/8" = 1'-0"

INITIAL CREST ▲ SPILLWAY WEIR COORDINATES

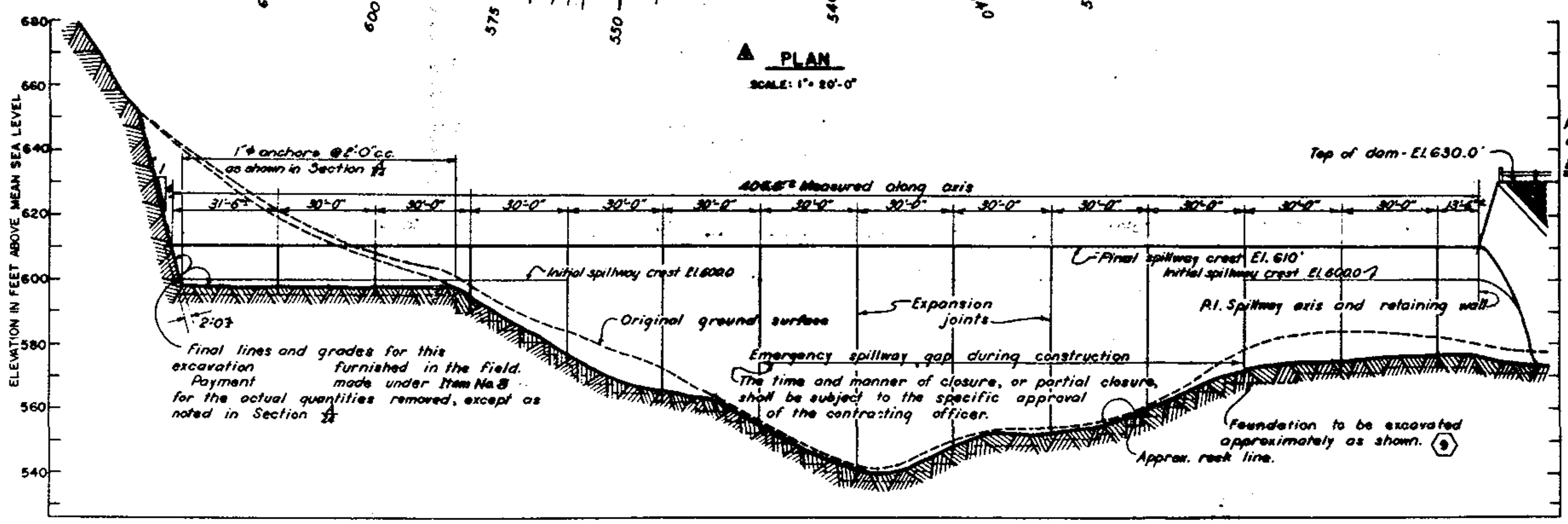
For parts constructed in 1940

SCALE: 1" = 5'-0"



▲ SECTION A/24

SCALE: 1/8" = 1'-0"



▲ DOWNSTREAM ELEVATION-DEVELOPED

SCALE: 1" = 20'-0"

NOTES

Concrete in the spillway shall be Class "B." Payment made under Item No.30.

No horizontal field joints shall be made in the spillway above El.595.0 except as noted.

The depth of foundation excavation for the spillway weir determined in the field, and payment made under Item No.9 for the actual quantities removed.

Figures in hexagons indicate item number, under which payment made.

Concrete surface at all expansion joints shall be uncoated.

See Sheet No.27 for details of V strips for all vertical expansion joints.

▲	1-13-41	Spillway elevation raised	O.B.M.	R.M.	J.W.
KEY	DATE	REVISIONS (Indicated by △)	REV. BY	CK. BY	AP. BY

RECORD DRAWINGS

CONNECTICUT RIVER FLOOD CONTROL

KNIGHTVILLE DAM

SPILLWAY

WESTFIELD RIVER, MASSACHUSETTS

IN 83 SHEETS — SCALE: 1 IN = 20 FT. — SHEET NO. 24

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1939

SUBMITTED: Harold B. Kinnison, SENIOR ENGINEER, HEAD, DESIGN SECTION

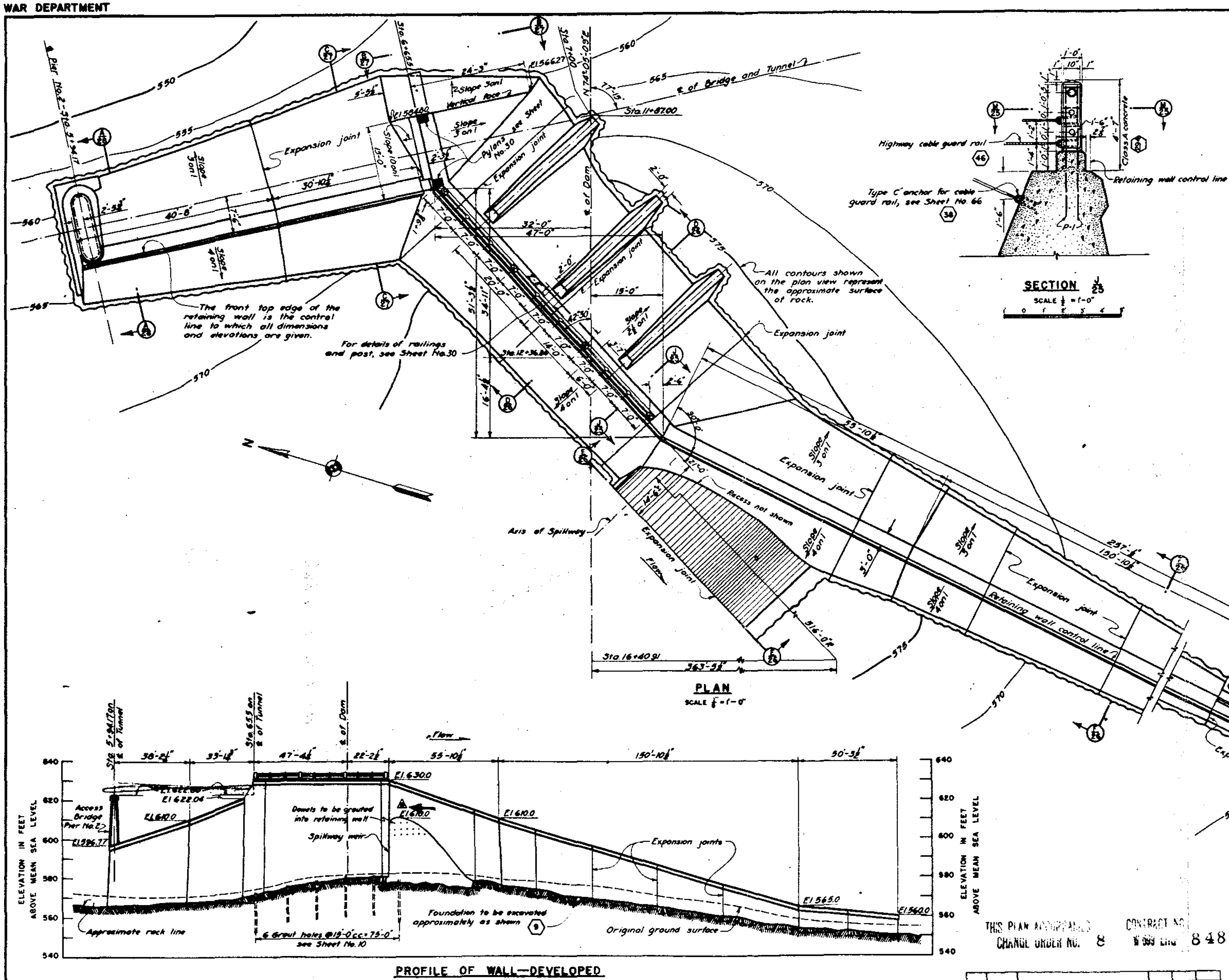
APPROVAL RECOMMENDED: T. J. Burns, PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV.

APPROVED: J. W. Bagley, LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

DESIGNED: A. P. Mumpower, ASSOC. ENGINEER

DRAWN BY: M.B.M. TRACED BY: J.B. CHECKED BY: M.M.

FILE NO. CT. 1-1261



NOTES

All concrete shown on this sheet shall be class B and will be paid for under Item No. 30 except as noted on Sheets No. 26 to 28 inclusive.

All outside corners of concrete in piers and abutment shall be chamfered as directed by the contracting officer.

The depth of foundation excavation for the retaining walls and bridge piers will be determined in the field. Payment will be made for the actual quantities excavated.

Figures in hexagons indicate Item number under which payment will be made.

For details of service bridge see Sheets No. 30 and 31.

Concrete surface at expansion joints shall be uncoated.

See Sheet No. 27 for details of V-strips for all vertical expansion joints.

Furnishing and installing the wall railing will be paid for under Item No. 40.

THIS PLAN ACCOMPANIES CHANGE ORDER NO. 8 CONTRACT NO. W 09 Eng 848

KEY	DATE	REVISION (Indicated by △)	REV. BY	CK. BY	AP. BY
△	1-13-41	Spillway elevation raised	G.D.M.	M.H.	J.S.B.

CONNECTICUT RIVER FLOOD CONTROL

KNIGHTVILLE DAM

RETAINING WALL NO. 1

WESTFIELD RIVER, MASSACHUSETTS.

IN 82 SHEETS — SCALE 1/8" = 1'-0" — SHEET NO. 25

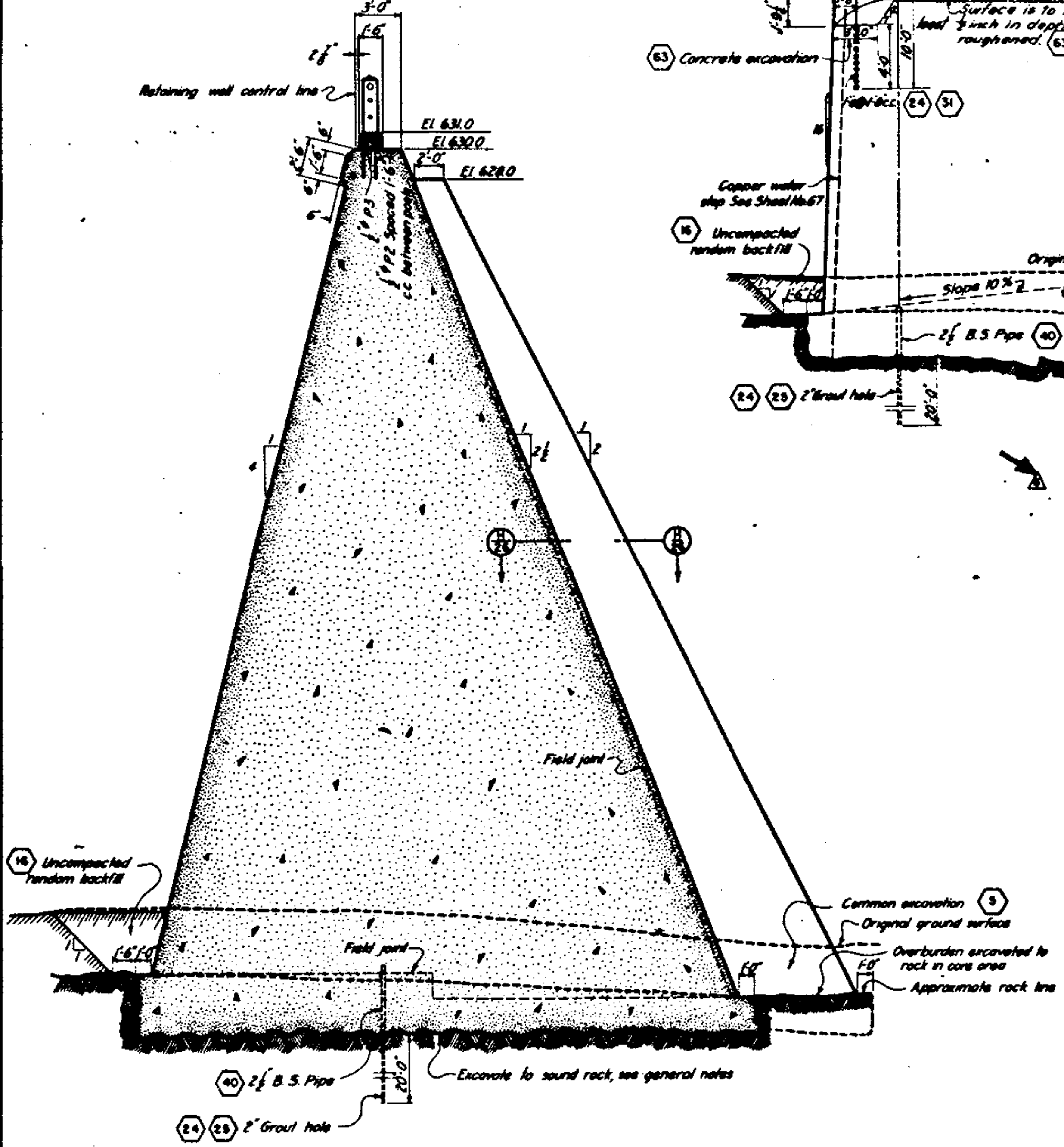
U.S. ENGINEER OFFICE, PROVIDENCE, R.I. JAN. 1939

SUBMITTED: SENIOR ENGINEER, HEAD, DESIGN SECTION — APPROVAL RECOMMENDED: T. S. Burns, PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV. — APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DESIGNED: G. D. Mu..., ASSOC. ENGINEER — DRAWN BY: M.S.H. — TRACED BY: P.J.S. — CHECKED BY:

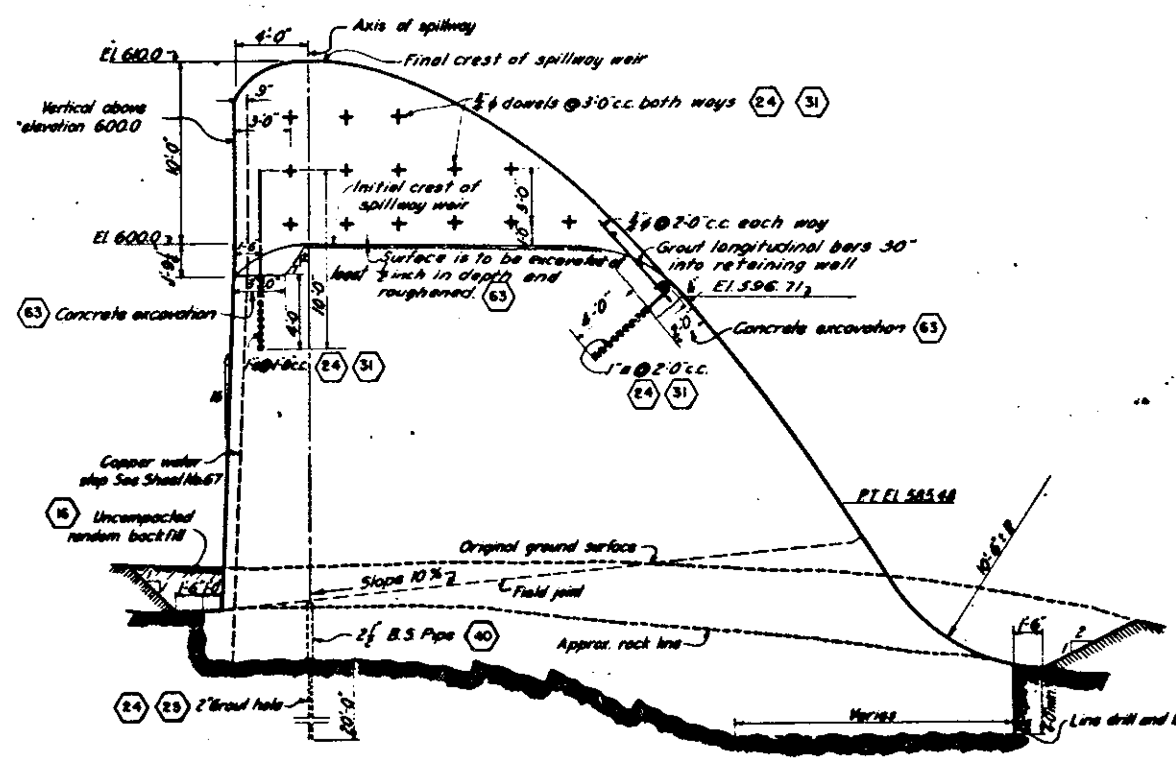
FILE NO. CT-1-1284

PRINTED JAN 15 1941



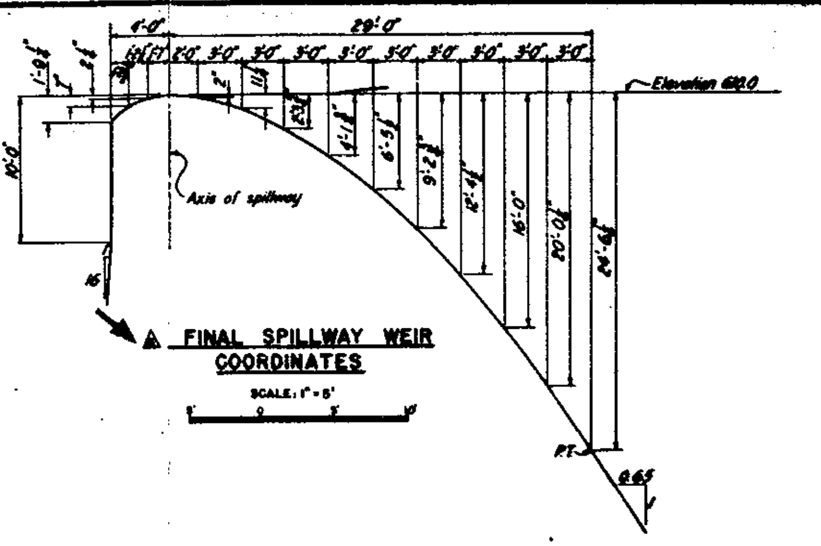
SECTION D/25

SCALE: ¼" = 1'-0"



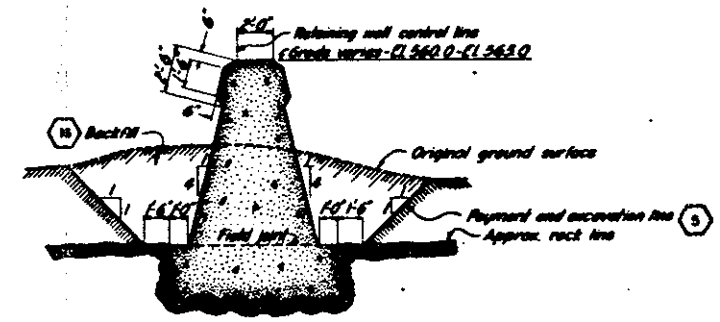
ELEVATION E/25

SCALE: ¼" = 1'-0"



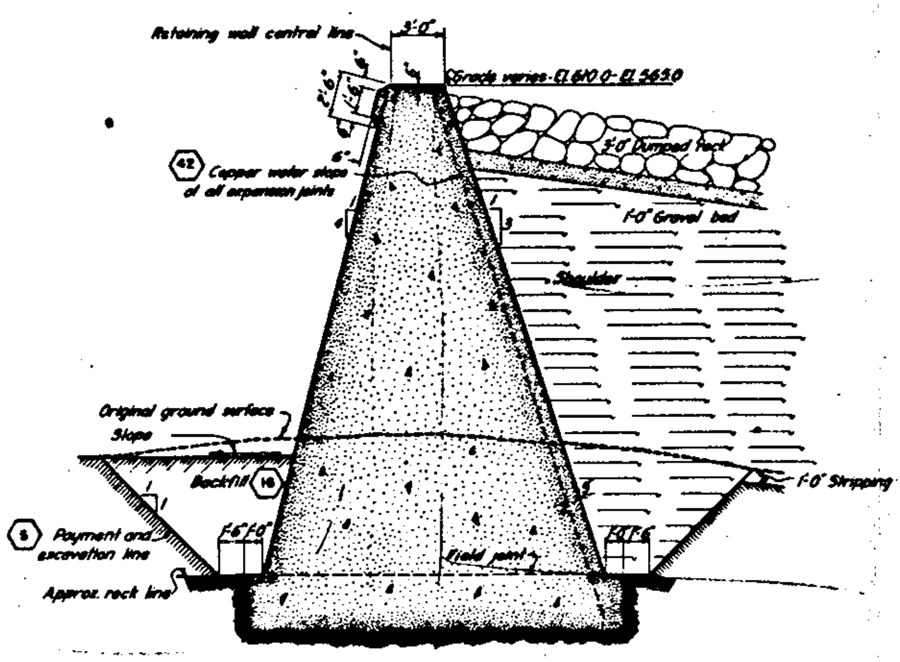
FINAL SPILLWAY WEIR COORDINATES

SCALE: 1" = 5'



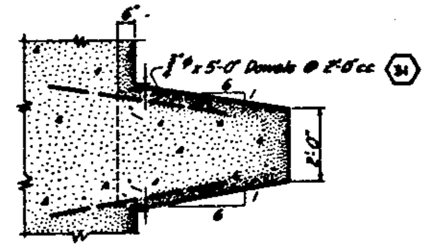
SECTION G/25

SCALE: ¼" = 1'-0"



SECTION F/25

SCALE: ¼" = 1'-0"



SECTION H/25

SCALE: ½" = 1'-0"

NOTE

For general notes, see notes on Sheet No. 25

THIS PLAN ACCOMPANIES CHANGE ORDER NO. 8

CONTRACT NO. W 699 ENG. 848

KEY	DATE	REVISION (Indicated by △)	REV. BY	OK BY	AP. BY
A	1/13/41	Spillway elevation revised			

CONNECTICUT RIVER FLOOD CONTROL

KNIGHTVILLE DAM

RETAINING WALL NO. 2

WESTFIELD RIVER MASSACHUSETTS

IN 62 SHEETS SCALE: 1/4 IN. = 1 FT. SHEET NO. 26

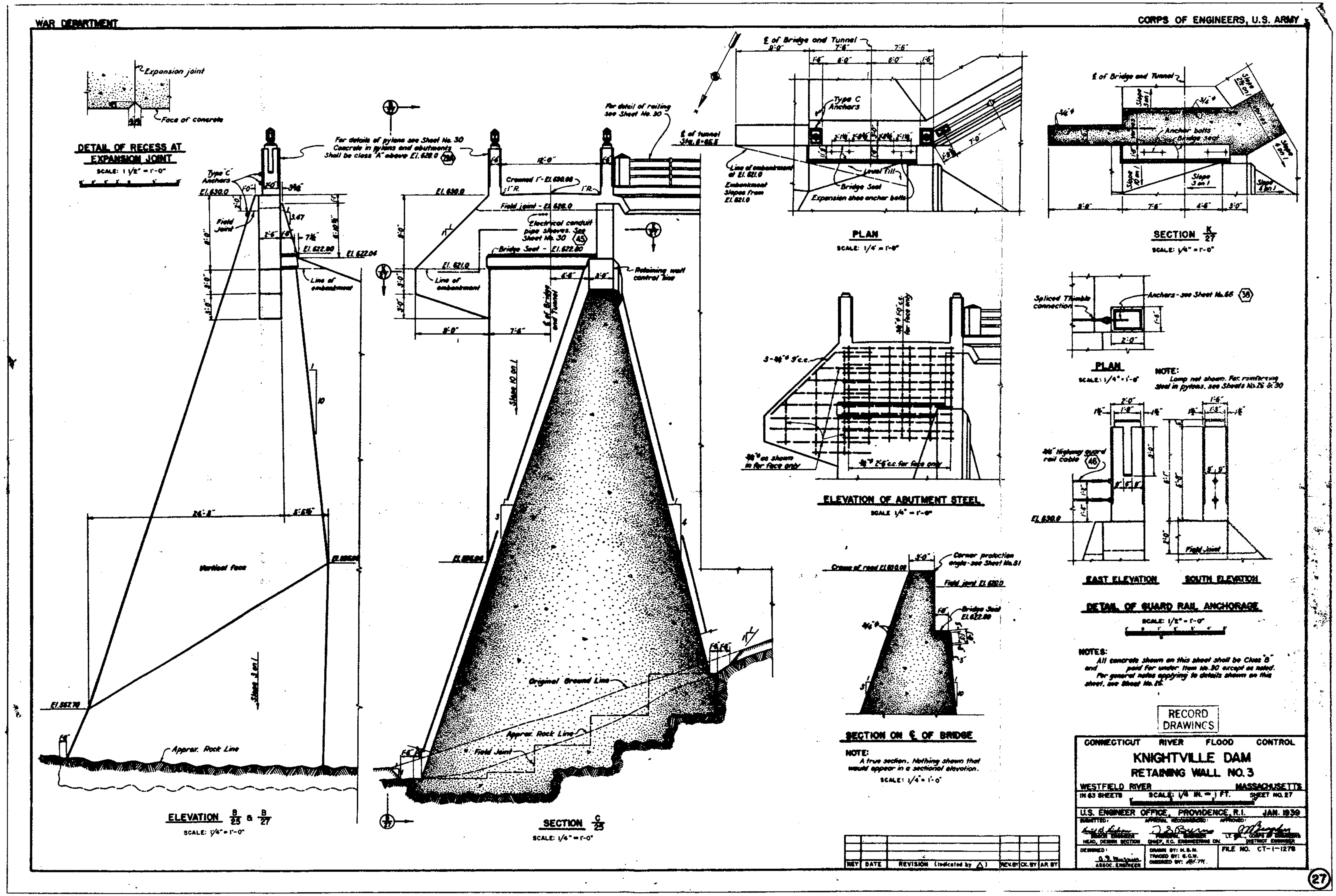
U.S. ENGINEER OFFICE, PROVIDENCE, R.I. JAN. 1939

SUBMITTED: SENIOR ENGINEER, HEAD DESIGN SECTION — APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF F.C. ENGINEERING DIV. — APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

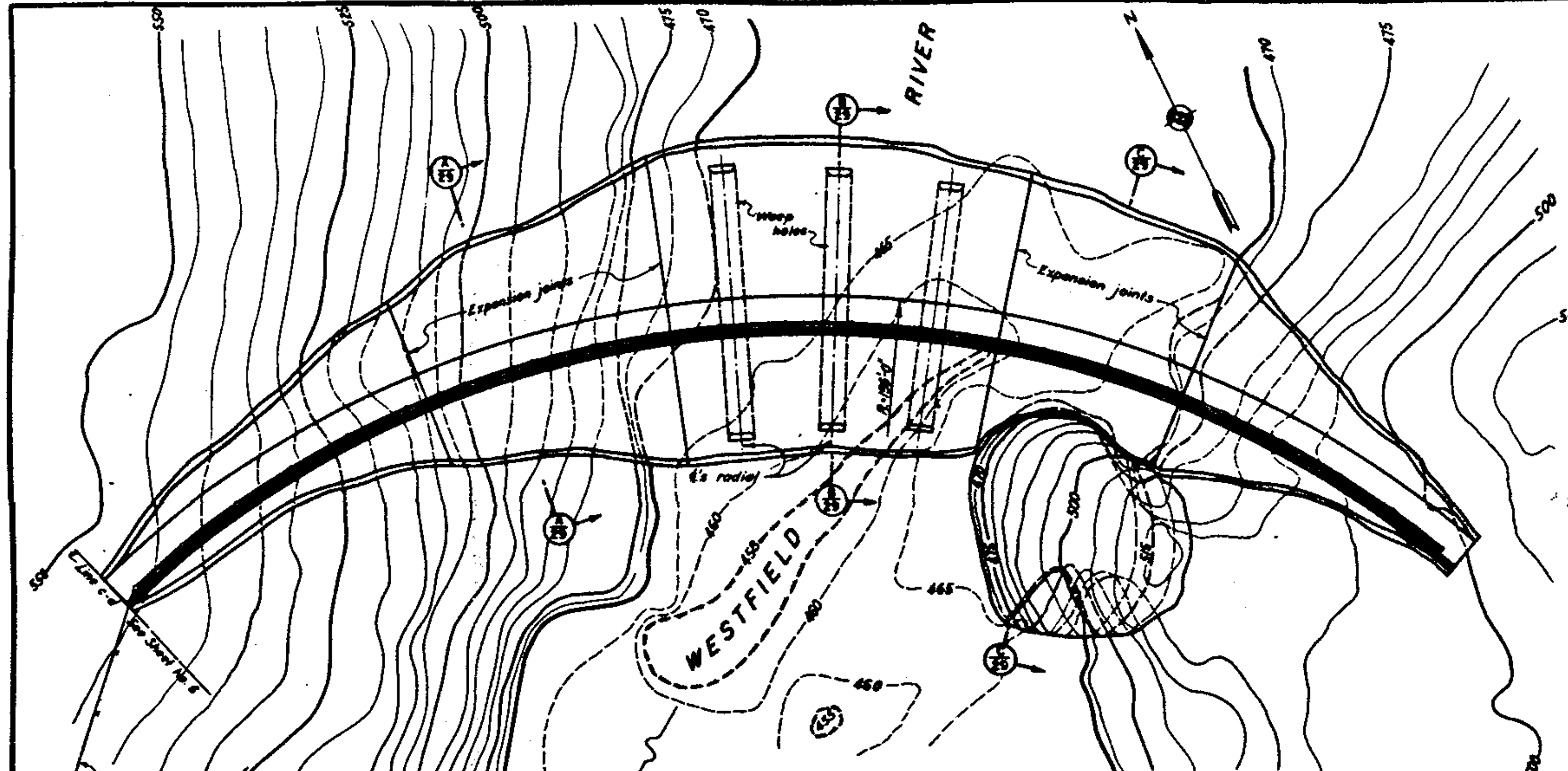
DESIGNED — DRAWN BY — TRACED BY — CHECKED BY

FILE NO. CT-1-1315

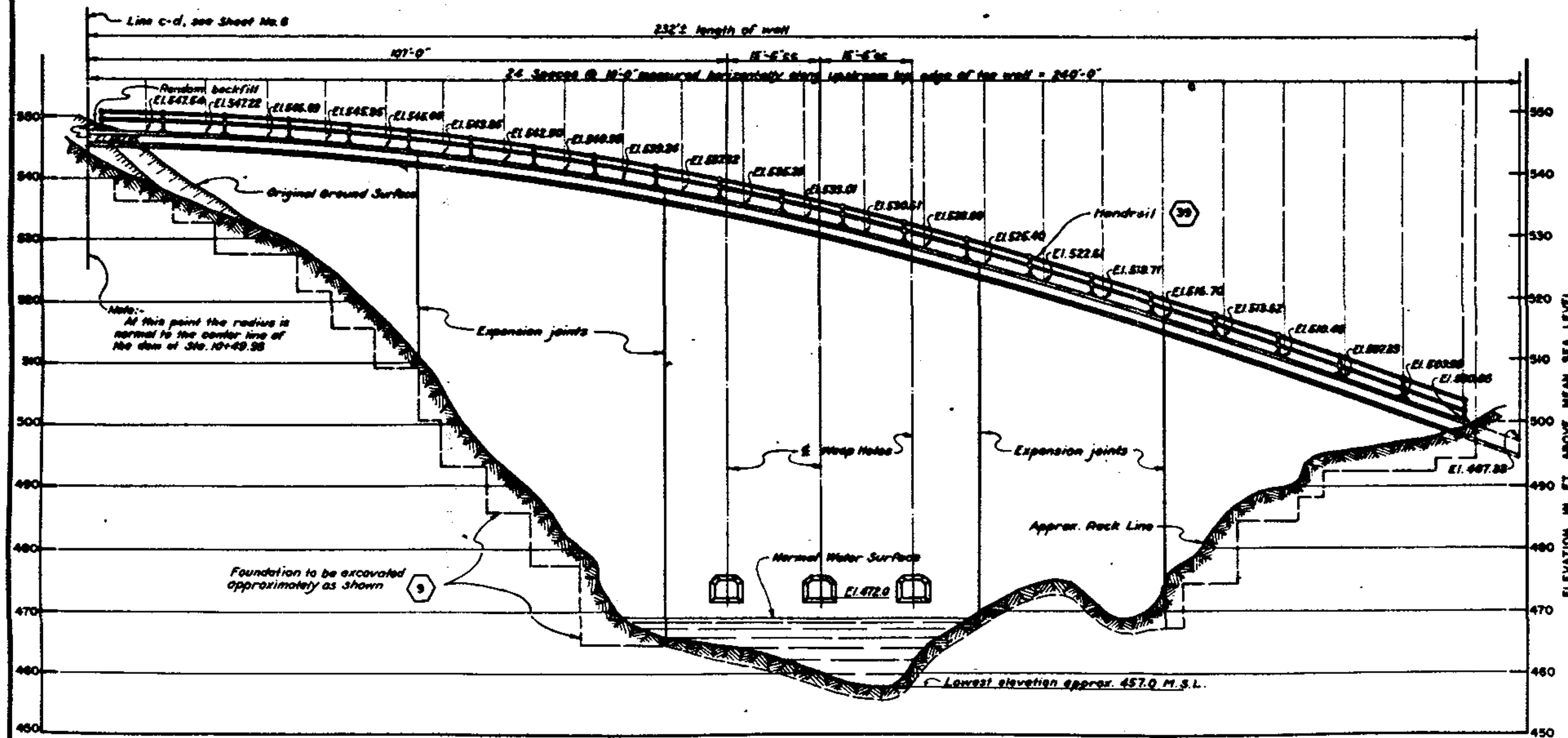
PRINTED JAN 13 1941



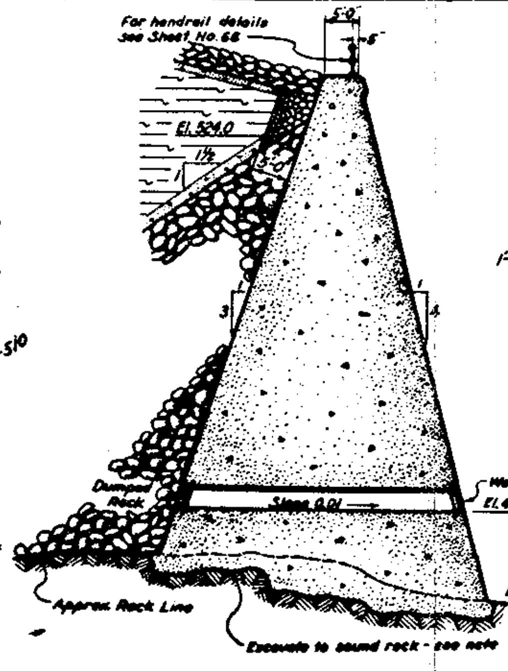
WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
DETAIL OF RECESS AT EXPANSION JOINT
SCALE: 1 1/2" = 1'-0"
Expansion joint
Face of concrete
For details of pylons see Sheet No. 30
Concrete in pylons and abutments Shall be class "A" above El. 628.0
For detail of railing see Sheet No. 30
Crowned 1' - El. 630.66
Field joint - El. 626.0
Electrical conduit pipe sleeves. See Sheet No. 30
Bridge Seat - El. 622.80
Retaining wall control line
Line of embankment
Vertical face
Slope 10 on 1
Slope 3 on 1
Original Ground Line
Approx. Rock Line
Field Joint
ELEVATION B/25 & B/27
SCALE: 1/4" = 1'-0"
SECTION C/25
SCALE: 1/4" = 1'-0"
PLAN
SCALE: 1/4" = 1'-0"
SECTION K/27
SCALE: 1/4" = 1'-0"
ELEVATION OF ABUTMENT STEEL
SCALE: 1/4" = 1'-0"
SECTION ON ℄ OF BRIDGE
NOTE:
A true section. Nothing shown that would appear in a sectional elevation.
SCALE: 1/4" = 1'-0"
Corner protection angle-see Sheet No. 81
Crown of road El. 630.66
Field joint El. 628.0
Bridge Seat El. 622.80
Spliced Thimble connection
Anchors-see Sheet No. 66
PLAN
SCALE: 1/4" = 1'-0"
NOTE:
Lamp not shown. For reinforcing steel in pylons, see Sheets No. 26 & 30
EAST ELEVATION
SOUTH ELEVATION
DETAIL OF GUARD RAIL ANCHORAGE
SCALE: 1/2" = 1'-0"
NOTES:
All concrete shown on this sheet shall be Class "B" and paid for under Item No. 30 except as noted.
For general notes applying to details shown on this sheet, see Sheet No. 26.
RECORD DRAWINGS
CONNECTICUT RIVER FLOOD CONTROL
KNIGHTVILLE DAM
RETAINING WALL NO. 3
WESTFIELD RIVER
MASSACHUSETTS
IN 63 SHEETS
SCALE: 1/4 IN. = 1 FT.
SHEET NO. 27
U.S. ENGINEER OFFICE, PROVIDENCE, R.I.
JAN. 1939
FILE NO. CT-1-1278
REV. DATE REVISION (Indicated by △) REV. BY CK. BY AR. BY
27



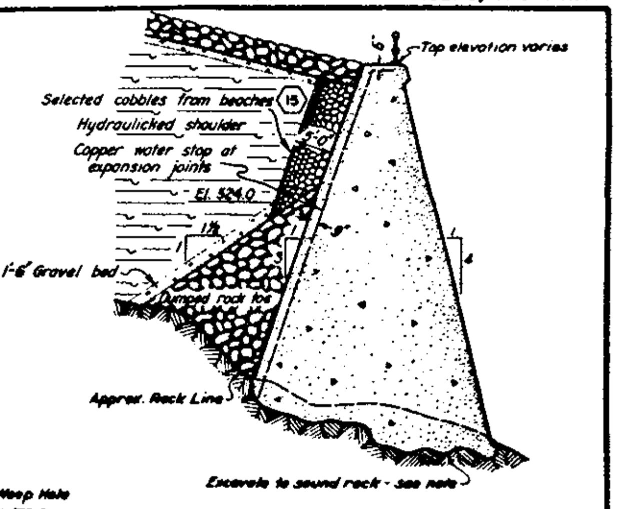
PLAN
SCALE: 1" = 10'



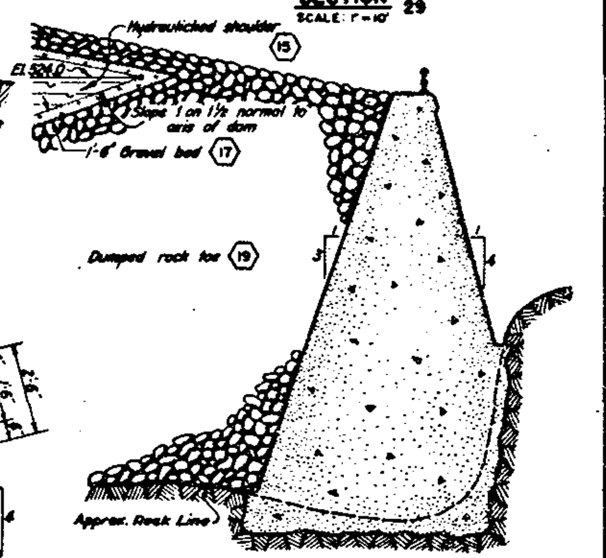
DOWNSTREAM ELEVATION — DEVELOPED
SCALE: 1" = 10'



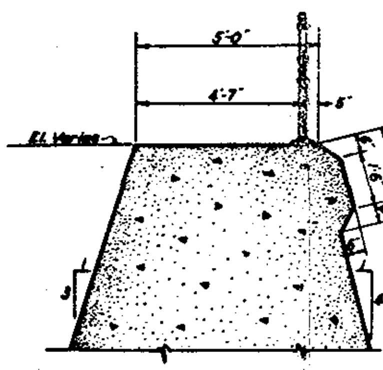
SECTION B/29
SCALE: 1" = 10'



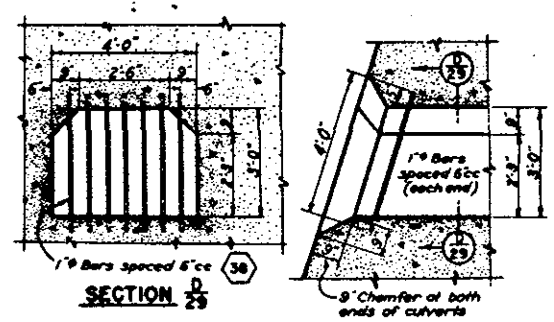
SECTION A/29
SCALE: 1" = 10'



SECTION C/29
SCALE: 1" = 10'



DETAILS OF TOP OF WALL
SCALE: 1/2" = 1'-0"



DETAILS OF WEEP HOLES
SCALE: 1/2" = 1'-0"

NOTES:

For location of concrete toe wall see Sheet No. 6.

Upstream top edge of toe wall shall coincide with surface of dam.

Figures in hexagons indicate item number under which payment will be made.

Concrete shall be Class B and will be paid for under Item No. 30

Expansion joints shall be spaced approximately as shown.

The depth of foundation excavation for the toe wall will be determined in the field. Payment will be made for the actual quantities excavated.

Expansion joints shall not be coated.

For detail of face of concrete at expansion joints see Sheet No. 27.

DATE	REVISION	REV. BY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

KNIGHTVILLE DAM
CONCRETE TOE WALL

WESTFIELD RIVER — MASSACHUSETTS

IN 75 SHEETS — SCALE: 1 IN. = 10 FT. — SHEET NO. 29

U. S. ENGINEER OFFICE, PROVIDENCE, R.I. — JAN. 1939

SUBMITTED: SENIOR ENGINEER, HEAD, DESIGN SECTION
APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV.
APPROVED: LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

DESIGNED: ASSOC. ENGINEER
DRAWN BY J.S.M.
TRACED BY G.C.M.
CHECKED BY

FILE NO. CT-1-1235